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16. Abstract
A method has been developed to strengthen existing deteriorating bridge slabs from underneath, thus eliminating disruption of traffic. A small portion of two structures near downtown Houston, Texas, was strengthened using this method. The effectiveness of this strengthening was evaluated by means of field measurements of deflection and strain under static wheel loading before and after strengthening.

The strengthening system was erected under traffic conditions with little difficulty. It reduced slab deflections and stresses by more than 50% and provided a significant increase in ultimate strength. The strengthening system has not healed the existing slab cracking and some type of surface sealing will be required to prevent further surface decomposition by exposure to contaminants.

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Evaluation of Bridge Slab Strengthening System

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Research Report Number 219-1F

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

SUMMARY

A method has been developed to strengthen existing deteriorating bridge decks from underneath, thus eliminating disruption of traffic. A small portion of two structures near downtown Houston, Texas, was strengthened using this method. This research study was conducted to evaluate the effectiveness of this strengthening system. Evaluation was accomplished by measuring deflections and strains under a static wheel load before and after strengthening. Attempts were made to use aerial photography in evaluating the strengthening system; however, this was not very successful.

The strengthening system was erected under traffic conditions with little difficulty. It reduced slab deflections and stresses by more than 50% and provided a significant increase in ultimate strength. The strengthening system has not healed the existing slab cracking and some type of surface sealing will be required to prevent further surface decomposition by exposure to contaminants.

RECOMMENDATION FOR IMPLEMENTATION

A new method has been developed to strengthen deteriorating bridge slabs from underneath, and thus eliminating disruption of traffic. It is recommended that this method of repair be considered when no suitable means of detouring traffic is available and removing and replacing a slab would cause severe traffic congestion and inconvenience to the travelling public.

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CHAPTER I

INTRODUCTION

Many older highway structures which were designed for 12 kip (5443.1 kg) wheel loads are now experiencing extensive deck distress and deterioration due to increased traffic loads. On heavily travelled facilities where adequate detours are not available, current repair methods require a disruption of traffic which results in high user costs. This report describes a method developed to strengthen these deteriorating slabs from underneath, thus eliminating the disruption of traffic.

Test Structures

The structures selected for the installation of this pilot strengthening project are structures 97 and 98 of the IH-45 interchange complex near downtown Houston (See Figures 1.1 and 1.2). The maximum daily traffic in this area is approximately 150,000 vehicles with peak hour traffic usually bumper to bumper. This elevated portion of IH-45 is approximately six tenths of a mile (0.96 km) in length and was completed in 1961.

These structures consist of rolled steel beams, continuous for two to three spans, with span lengths averaging approximately 70 feet (21.34 m). The beams are 33 WF 130 spaced on about 8 foot (2.44m) centers with cover plates over the supports. The slab is $6\frac{1}{2}$ inches (16.5 cm) thick and lightweight concrete was used in a majority of the spans. The slab was designed using 1957 AASHO Specifications for Highway Bridges which provided for a design wheel load of 12 kips (5443.1 kg). Under the current standards the slab is under-designed by approximately 26 percent.

Objectives

The overall objective of this study was to thoroughly evaluate this pilot project to determine the practicality of the repair method and provide a tangible basis for decision making involving future repair work of this type. Specific objectives of this research study were as follows:

1. Verify the assumptions and approximations for the theoretical analysis used to design the strengthening system.
2. Determine if the system relieves slab distress and prevents further deterioration.
3. Determine whether or not slab deflections are the primary cause of the deck deterioration.
4. Determine if the strengthening system can be installed as required and is within the capabilities of the average highway contractor.

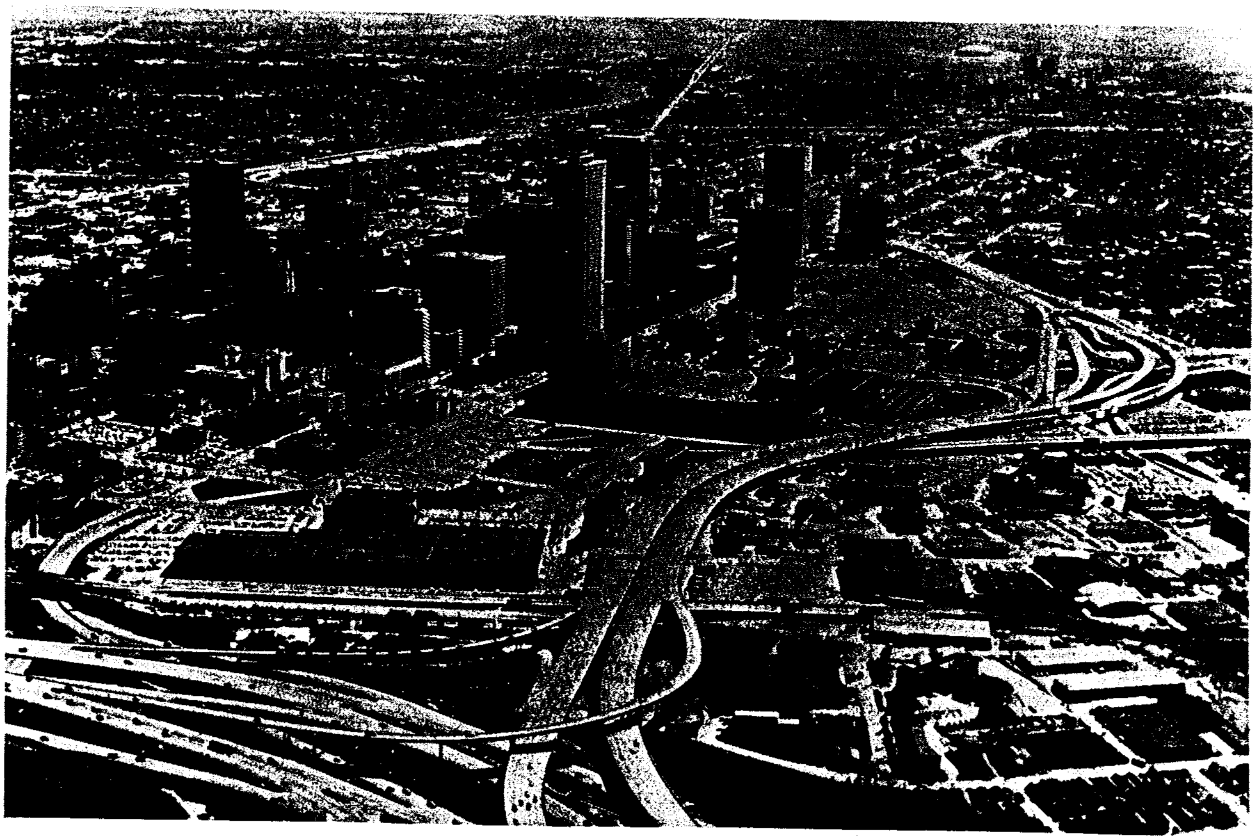


Figure 1.1. I.H. 45 Near Downtown Houston

Figure 1.2. Structures 97 and 98, I.H. 45

CHAPTER II

CONDITION OF EXISTING BRIDGE SLABS

In 1972 maintenance personnel reported that extensive repair work was being required on the slabs of structures 97 and 98. As a result of these reports an extensive investigation of structure 97 was undertaken to determine the extent of this deck deterioration. The deck was subjected to a thorough visual inspection, a large area of deck was inspected for delamination, and several core samples were taken for examination and testing. The following results of this investigation were reported by Mr. M. U. Ferrari of the Texas State Department of Highways and Public Transportation:

1. The no traffic or shoulder lane of the lightweight slabs exhibited no visible cracking on the top surface with some widely spaced hairline cracking at random intervals on the bottom surface.
2. The travelled lanes showed closely spaced pattern or map cracking over the entire lightweight surface with "working" tension cracks on the underside. There was some evidence of water migration through the deck.
3. Surface cracking in the hard-rock concrete spans

was in evidence although less in extent than in the lightweight spans.

4. Examination of the cores showed lightweight slab thicknesses from 6-5/16 inches to 6-5/8 inches (16.02-16.84 cm). Bottom tension cracking on some of the cores, particularly the shorter ones, was traced up to the neutral axis of the slab, with cracking passing through some of the lightweight aggregate. Depth of the hard-rock cores ranged from 6-7/8 inches to 6-15/16 inches (17.48-17.63 cm). No noticeable corrosion was found on the reinforcing steel even though some water pockets or voids were in evidence in the concrete around the steel perimeter.

5. The measured clear depth of the top layer of the reinforcing steel was greater than specified. This condition, in company with slab thicknesses less than specified, places the top and bottom layers of reinforcing steel closer together than intended. Clear distance between top and bottom layers of reinforcing steel ranged from 2.1 inches to 2.5 inches (5.33-6.35 cm) for lightweight concrete and 2.4 inches to 2.45 inches (6.10-6.22 cm) for the hard-rock concrete.

6. Chemical tests on the lightweight cores showed pro-

nounced carbonation from the upper surface down to 3/4 inch (1.90 cm) and deeper along cracks (a reflection of high water - cement ratio paste).

7. The petrography investigation revealed the following:
   a. Entrapped or free water.
   b. High water - cement ratio.
   c. No entrained air.
   d. Three to five percent entrapped air.
   e. Vertical cracking.
   f. Pronounced bleed channels.
   g. Secondary compound present in many of the voids (calcium - sulfate silicate or calcium - sulfate aluminate - probably from the aggregates).
   h. Complete hydration of all cement particles supporting high water - cement ratio.
   i. Highly porous paste (encouraging carbonation).
   j. Some specimens showed voids and lack of consolidation or washed out paste.

8. Compression tests were as follows:

Cores	Lightweight	
1	4,014 psi	(27.7 MPa)
2	3,714 psi	(25.6 MPa)
3	4,544 psi	(31.4 MPa)

4	4,253 psi	(29.3 MPa)
5	4,363 psi	(30.1 MPa)
	<u>Hard-rock</u>	
12	3,554 psi	(24.5 MPa)
13	4,412 psi	(30.4 MPa)

Figures 2.1 through 2.6 are photographs showing the condition of slabs on structures 97 and 98.

Figure 2.1. Structure 97 at Time of Strengthening
(Strengthened Area is Between Dashed Lines)

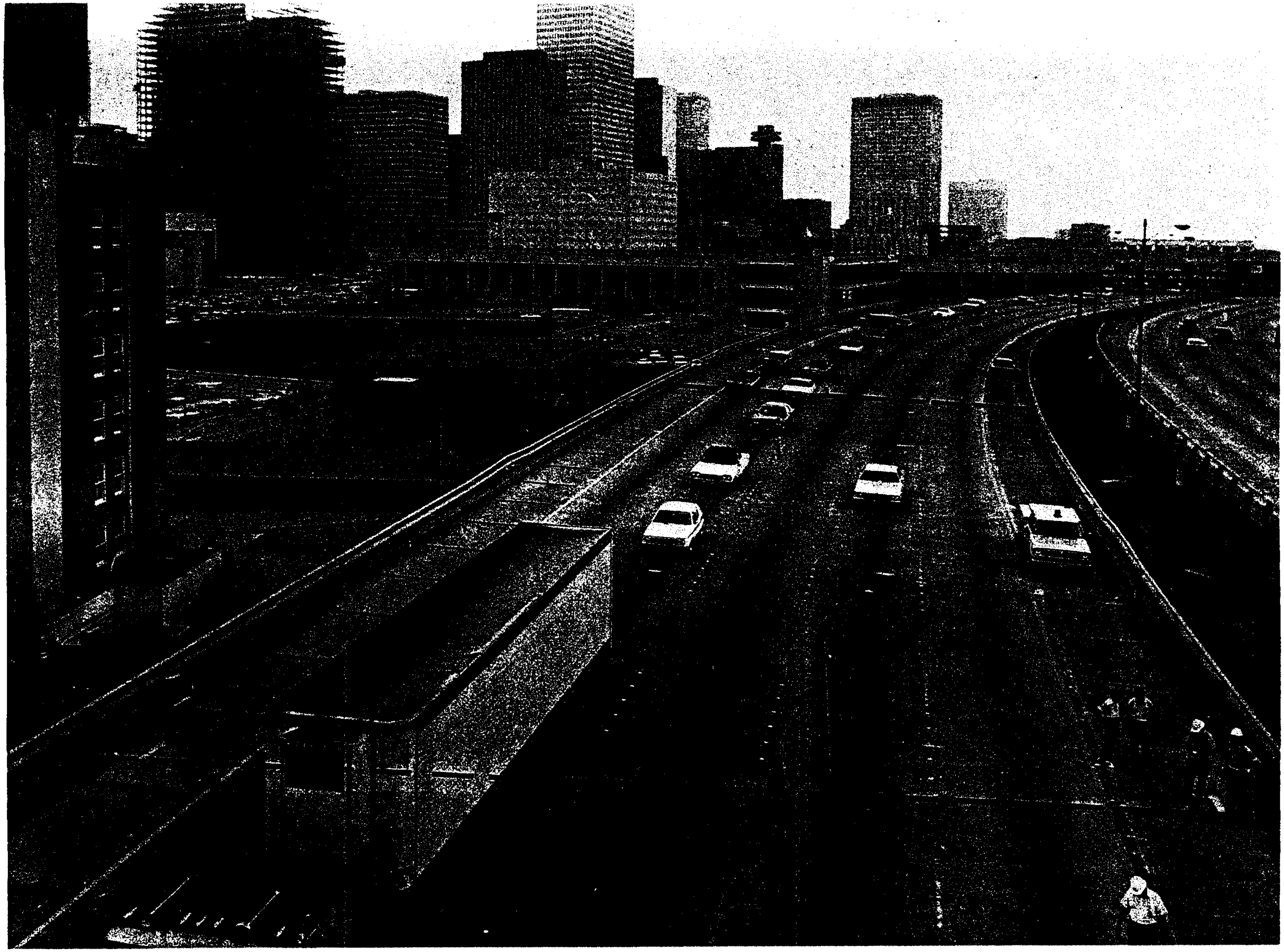


Figure 2.2. Structure 98 at Time of Strengthening
(Strengthened Area is Between Dashed Lines)

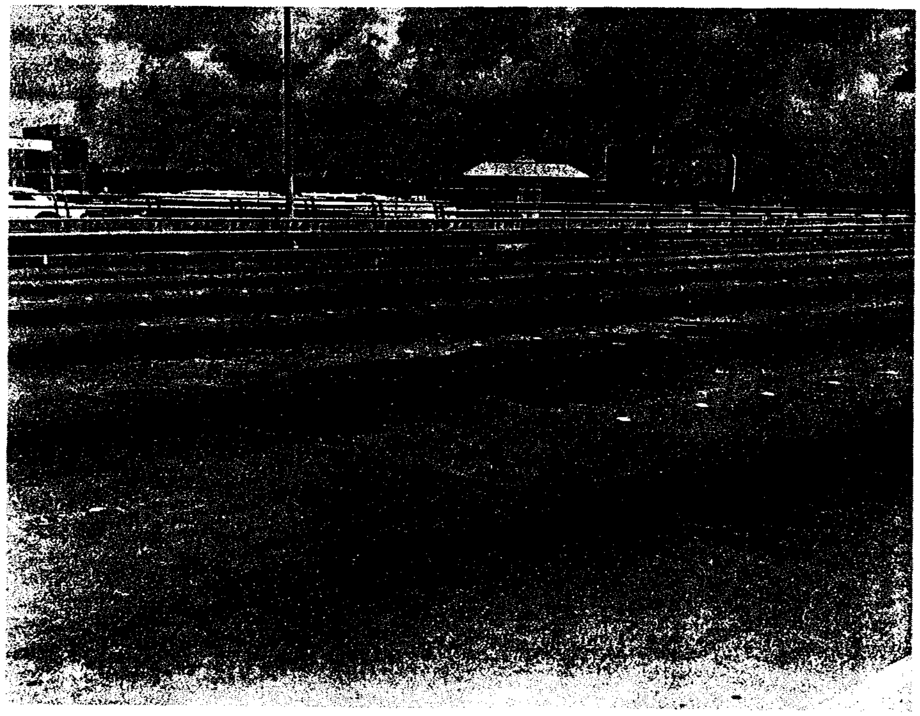


Figure 2.3. Slab Condition, Structure 98

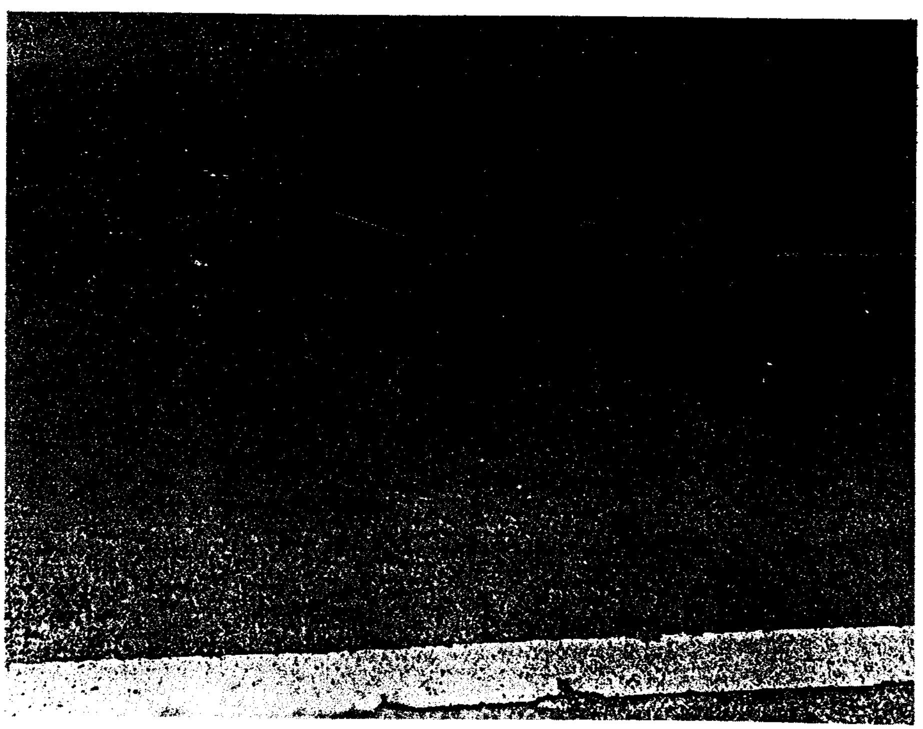


Figure 2.4. Crack Pattern, Structure 97

Figure 2.5. Outside Traffic Lane and Shoulder, Structure 98

Figure 2.6. Condition of Adjacent Slab Placements, Structure 98

CHAPTER III

DESIGN OF SLAB STRENGTHENING SYSTEM

Design Approach

Several methods of repairing the deteriorated decks were considered. Included were the following:

1. Seal the deck with a rubber formula and overlay with asphalt.
2. Chip the slab down to the steel and overlay with concrete.
3. Remove and replace the slabs.
4. Strengthen the slabs with a system of supporting beams under the slab.

Due to the heavy traffic carried by the structure, and since no suitable means of detouring traffic is available, removing and replacing the deck or overlaying the surface with concrete would cause severe traffic congestion and inconvenience to the travelling public. It was decided that this type of repair was to be used only as a last resort. An asphalt overlay was ruled out because it was believed that this would only treat the slab cosmetically, and further slab damage would not be visible until it was too late.

Results of the investigation of the condition of Structure 97 indicated that a significant amount of strength remained in the slab and if the live load stresses could be reduced, the service life of the slab could be significantly extended. It was, therefore, decided to strengthen a portion of the deteriorating structure using a beam support system and evaluate its strengthening effects.

The approach taken to reduce the live load slab stresses was one of reducing the slab deflection, thus reducing the amount of live load carried by the slab. It was decided that a deflection reduction of approximately 50% would be necessary to help the deteriorating slab to any significant degree. After several trial designs, using a variety of beam arrangements, a grid system of beams was found to give the greatest benefit to the slab in terms of overall deflection reduction.

Design Assumptions

The following assumptions were made in order to provide a method by which a designer could select supporting beam sizes and spacing with reasonable accuracy without resorting to a rigorous mathematical analysis:

1. The slab configuration will conform to the deformation of the supporting grid beam system.
2. There is no composite action between the slab and the supporting grid system.
3. The supporting grid system is simply supported at

the ends.

4. For calculating stringer deflection, the wheel load will be uniformly distributed over its entire length.

5. For calculating beam deflection, the wheel load will be considered concentrated at mid-span of the beam.

6. The modulus of elasticity of the lightweight slab is 2.0×10^6 psi (13.8×10^6 kN/m^2).

7. The effective width of slab for calculating slab deflections is equal to the wheel distribution width in accordance with AASHTO Standard Specifications, Section 1.3.2(c), Case A; i.e.

$$b = 8L \,/\, (L + 2)$$

where: b = Distribution width, ft

L = Girder spacing, ft

To calculate slab deflection over a stringer, the full distribution width, b, is used.

To calculate slab deflection over a floorbeam, the effective slab width is $b' = Kb$, where:

$$K = 1 - \left[E_sI_s \,/\, (E_sI_s + E_cI_c)\right]^{3/2}$$

E_sI_s = Stiffness properties of steel floorbeam

E_cI_c = Stiffness properties of the concrete slab with effective width, b.

Design Method

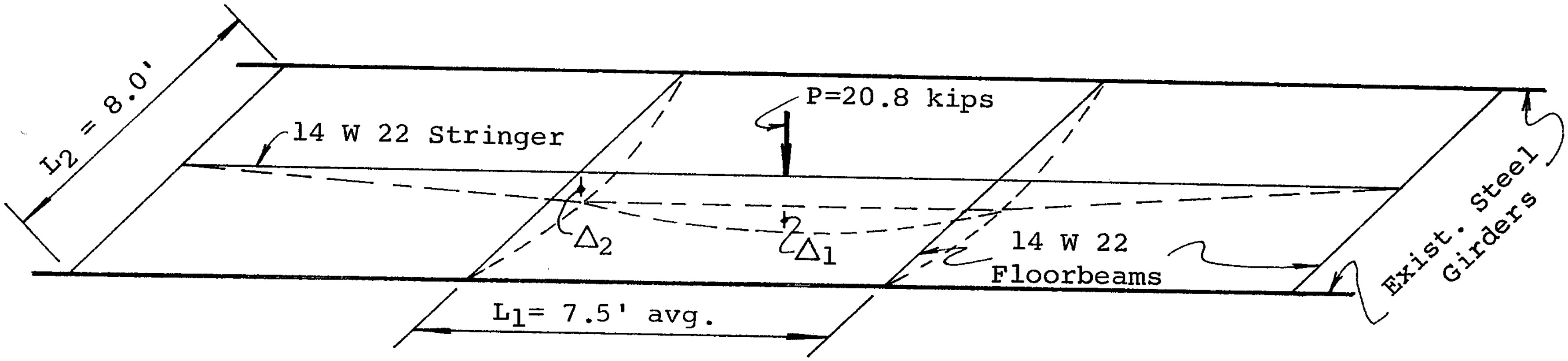
The method used to analyze the system of grid beams is one of equating slab and beam deflections, then solving for the percent of load carried by each. If the percent of load carried by the supporting beam is represented by the factor "C", then the percent carried by the slab is (1-C). Based on the assumption that the beam and slab deflect together, the slab deflection will be equated to the beam deflection in terms of "C" and the value of "C" determined.

Through a trial and error solution, it was found that a grid beam system of 14 W 22 beams gave a deflection benefit factor of approximately 50%. The following sample calculations are based upon the use of these beams. Concentrated loads, P, will be placed as shown in Figure 3.1 and the resulting deflections calculated.

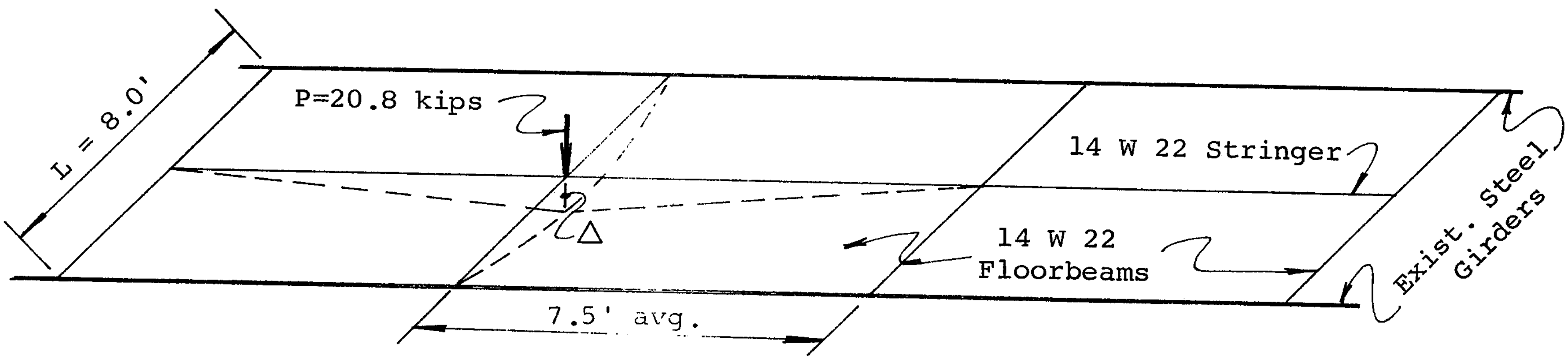
Calculations for Load on Stringer

Assume that the portion of the load P carried by the stringer is CP = 20.8(C) and is uniformly distributed throughout the stringer span (Figure 3.2a). The stringer support reactions acting on the centers of floorbeams are assumed as concentrated loads (Figure 3.2b).

$$\Delta_1 = \frac{5w\ L_1^3}{384\ E_s I_s} = \frac{5 \times (20.8C) \times (7.5)^3 \times (12)^3}{384 \times 29{,}000 \times 198}$$



(a) Load on Stringer



(b) Load on Floorbeam

Figure 3.1. Location of Design Loads

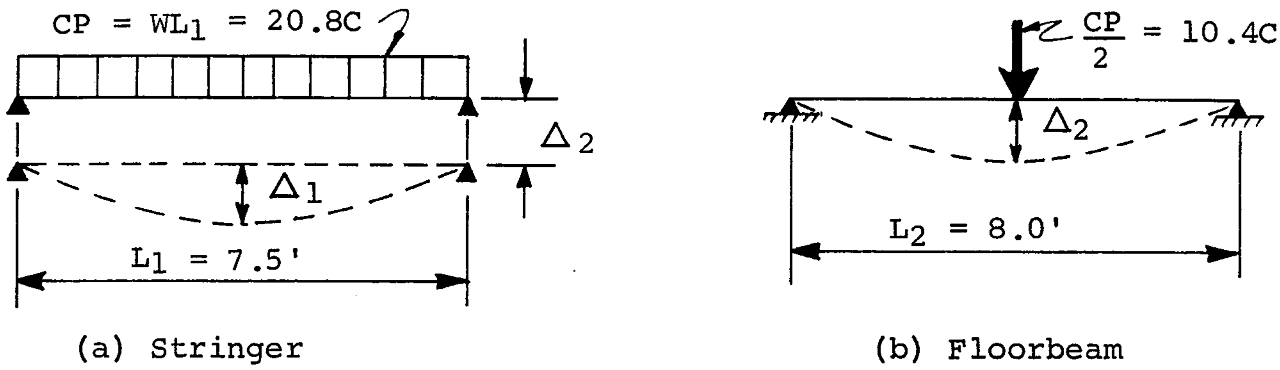


Figure 3.2. Stringer and Floorbeam Deflection

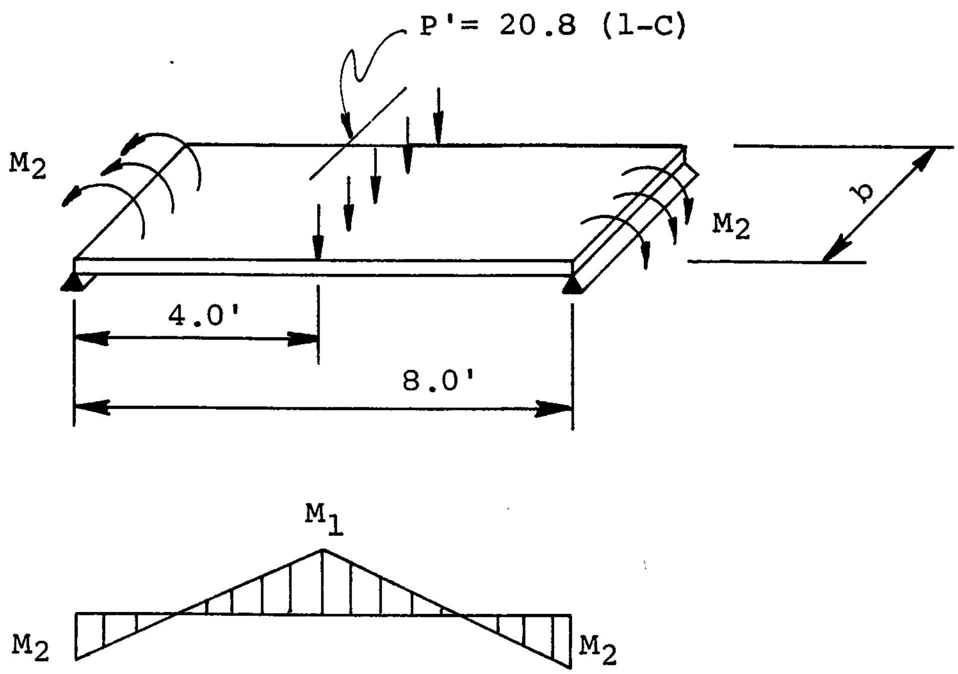


Figure 3.3. Load Distribution for Slab Deflection

$$\Delta_1 = 0.0344\ C\ (\text{in.})$$

$$\Delta_2 = \frac{(10.4\ C)L_2^3}{48\ E_s I_s} = \frac{10.4\ C \times (8.0)^3 \times (12)^3}{48 \times 29{,}000 \times 198}$$

$$\Delta_2 = 0.0334\ C\ (\text{in.})$$

$$\Delta_1 + \Delta_2 = 0.0678\ C \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ (1)$$

To calculate the slab deflection a concentrated load P', equal to 20.8(1-C), is placed at the center of an interior span of transverse slab and distributed over a width "b" as shown in Figure 3.3.

Using moment coefficients for a series of continuous spans,

$$M_1 = 0.172\ P'\ L_2$$

$$M_2 = 0.078\ P'\ L_2$$

the slab deflection may then be written:

$$\Delta_{slab} = \frac{P'\ L_2}{96\ E_c I_c}\left(2\ L_2^2 - \frac{12\ L_2\ M_2}{P'}\right)$$

substituting for M_2

$$\Delta_{slab} = \frac{P'\ L_2}{96\ E_c I_c}\left(2\ L_2^2 - \frac{12\ L_2 \times 0.078\ P' L_2^3}{P'}\right)$$

$$\Delta_{slab} = \frac{0.01108\ P'\ L_2^3}{E_c I_c} \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ (2)$$

where $I_c = \frac{1}{12}\ b\ d^3$ and $b = \frac{8\ L_2}{(L_2 + 2)}$

$$I_c = \frac{1}{12} \times \frac{8 \times 8.0}{(8.0 + 2)} \times 12 \times (6.5)^3 = 1{,}757.6\ \text{in}^4$$

$$\Delta_{slab} = \frac{0.01108 \times 20.8\ (1-C)\ (8.0)^3 \times (12)^3}{2,000 \times 1,757.6}$$

$$\Delta_{slab} = 0.0580\ (1-C)\ . . . .(3)$$

Equating Equations (1) and (3),

$$0.0678\ C = 0.0580\ (1-C)$$

$$C = 0.46$$

Thus 46% of the applied load is carried by the supporting stringer, near enough to the desired 50%.

Calculations for Load on Floorbeam

The floorbeam deflection due to a load CP located at mid-span is:

$$\Delta_{Fl.\ Bm.} = \frac{CPL^3}{48\ E_sI_s} = \frac{C \times 20.8 \times (8.0)^3 \times (12)^3}{48 \times 29,000 \times 198}$$

$$\Delta_{Fl.\ Bm.} = 0.0668\ C\ . . . . (4)$$

For slab deflection the load is distributed over a width b':

$$b' = (K)\ \frac{8L}{(L + 2)}$$

where

$$K = 1 - \left(\frac{E_sI_s}{E_sI_s + E_cI_c}\right)^{3/2}$$

$$K = 1 - \left(\frac{29,000 \times 198}{29,000 \times 198 + 2,000 \times 1,757.6}\right)^{3/2}$$

$$K = 0.51$$

then

$$b' = 0.51 \times \frac{8 \times 8.0}{(8.0 + 2)} = 3.26' = 39.1''$$

Using similar Equation (2) from previous calculation:

$$\Delta_{slab} = \frac{.01108\ P'\ L^3}{E_c I_c'}$$

where

$$I_c = \frac{1}{12}\ b'd^3 = \frac{1}{12} \times 39.1 \times (6.5)^3 = 895.3\ in^4$$

$$\Delta_{slab} = \frac{.01108 \times 20.8\ (1-C) \times (8.0)^3 \times (12)^3}{2{,}000 \times 895.3} =$$

$$\Delta_{slab} = 0.114\ (1-C)\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ (5)$$

Equating (4) and (5):

$$0.0668\ C = 0.114\ (1-C)$$

$$C = 0.63$$

Thus 63% of the applied load is carried by the floorbeam.

CHAPTER IV

CONSTRUCTION

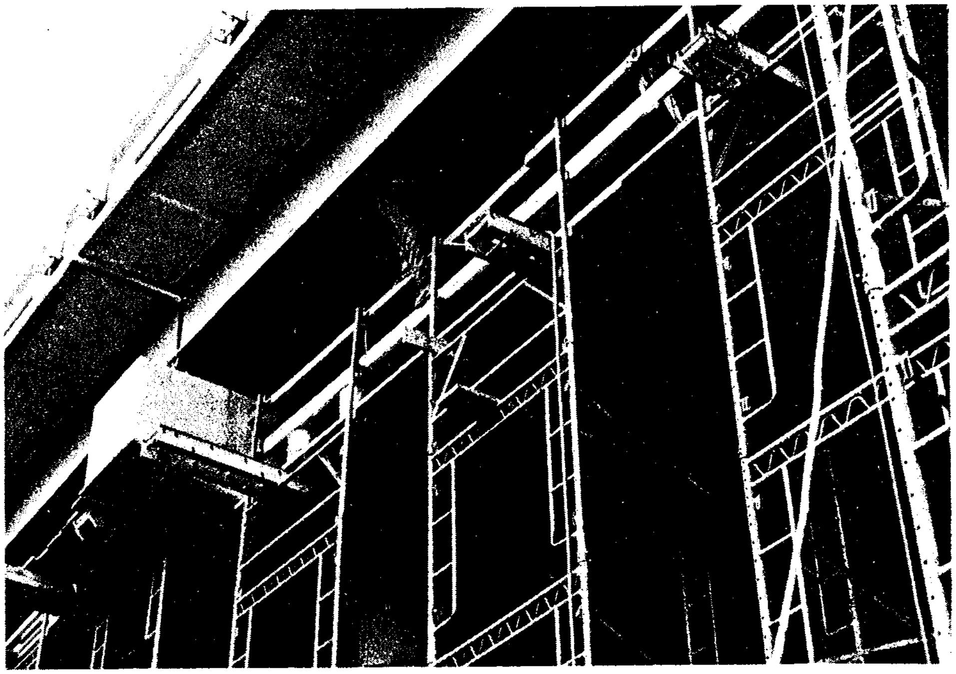
A work platform underneath the bridge deck was required since this work was to be accomplished without any disruption of the traffic above. The contractor provided enough materials for a platform under approximately one-half of the portion of each structure to be strengthened. This minimized the time required for erection and dismantling of the platform. The work platform is pictured in Figure 4.1.

To ensure good contact between the slab and grid beam system shim plates were placed on top of the beams and the space between the beams and slab filled with a stiff epoxy grout (details of the strengthening system are shown in Appendix A and the specifications for epoxy grout in Appendix B). Plans called for the shims to be hand driven between the slab and beams; however, this was revised to permit the contractor to preweld them to the beams (see Figure 4.2) and place the epoxy grout on top of the beams prior to jacking them into place. This eliminated the tedious work of placing the shims and packing the grout by hand after the beams were in place.

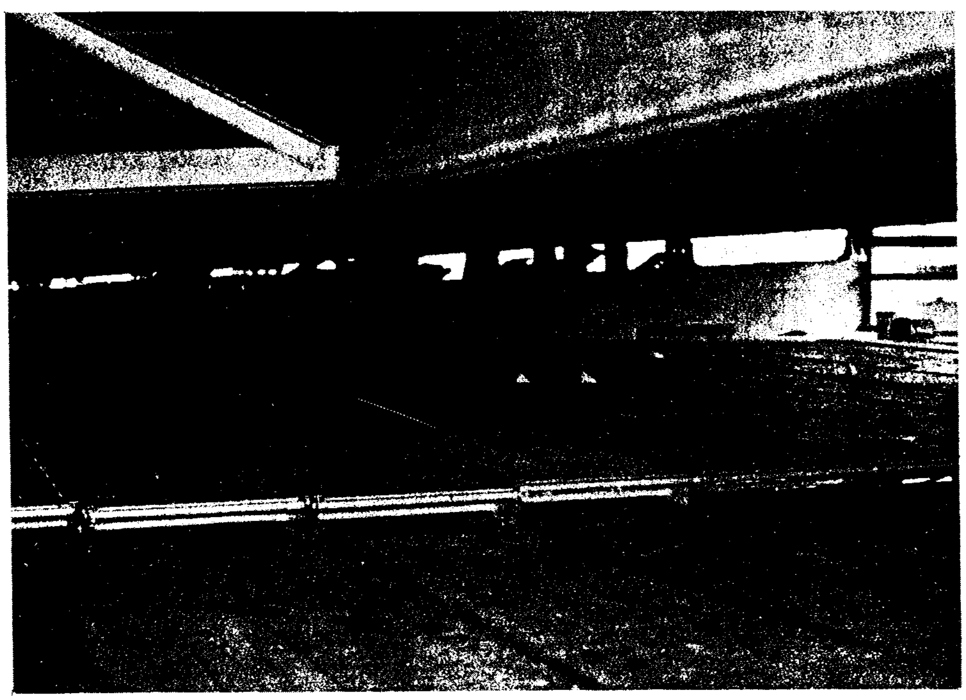
Small hand operated hydraulic jacks were used to position the beams (see Figure 4.3). The pressure gage for each jack was calibrated to read directly in pounds. The floorbeams were jacked into position first with a maximum load of 1000 pounds

(453 kg). The stringers were then jacked into place using approximately one-half the load used for the floorbeams. This method produced a very tight fit between the slab and shim plates. Figures 4.4 through 4.8 show the grid system in place.

One of the objectives of this project was to find out if the designed strengthening system could be installed by the average contractor. The ease with which this system can be installed is reflected in the short time required to erect the steel after it was delivered to the job site. The more than 750 pieces of steel were erected and painted in less than two months.



(a) Platform Support



(b) Work Platform

Figure 4.1. Working platform

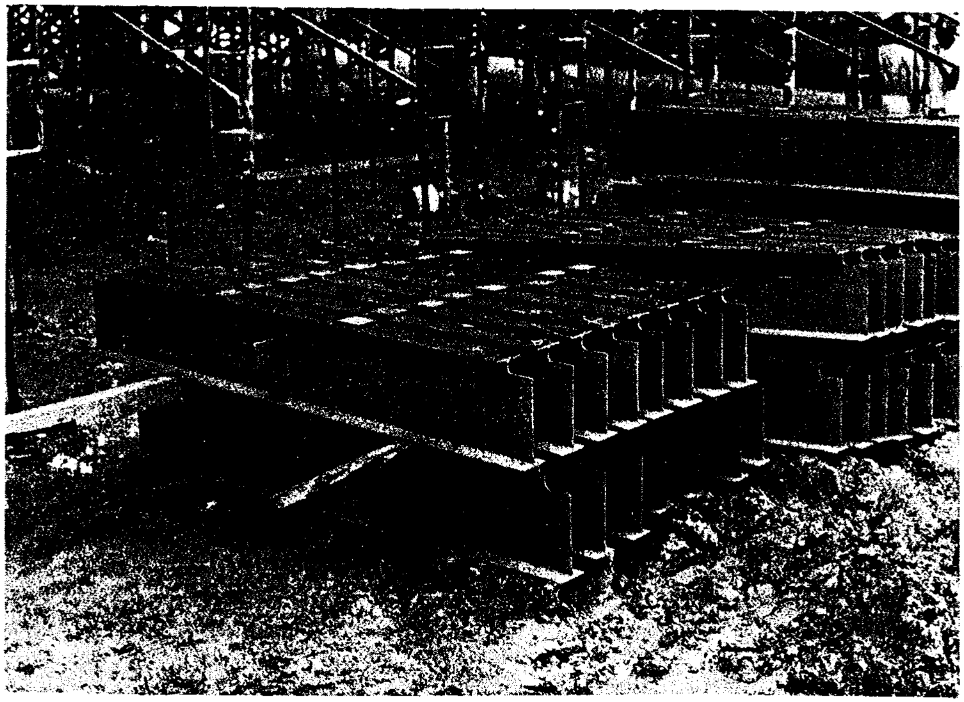


Figure 4.2. Stockpile of Grid Beams

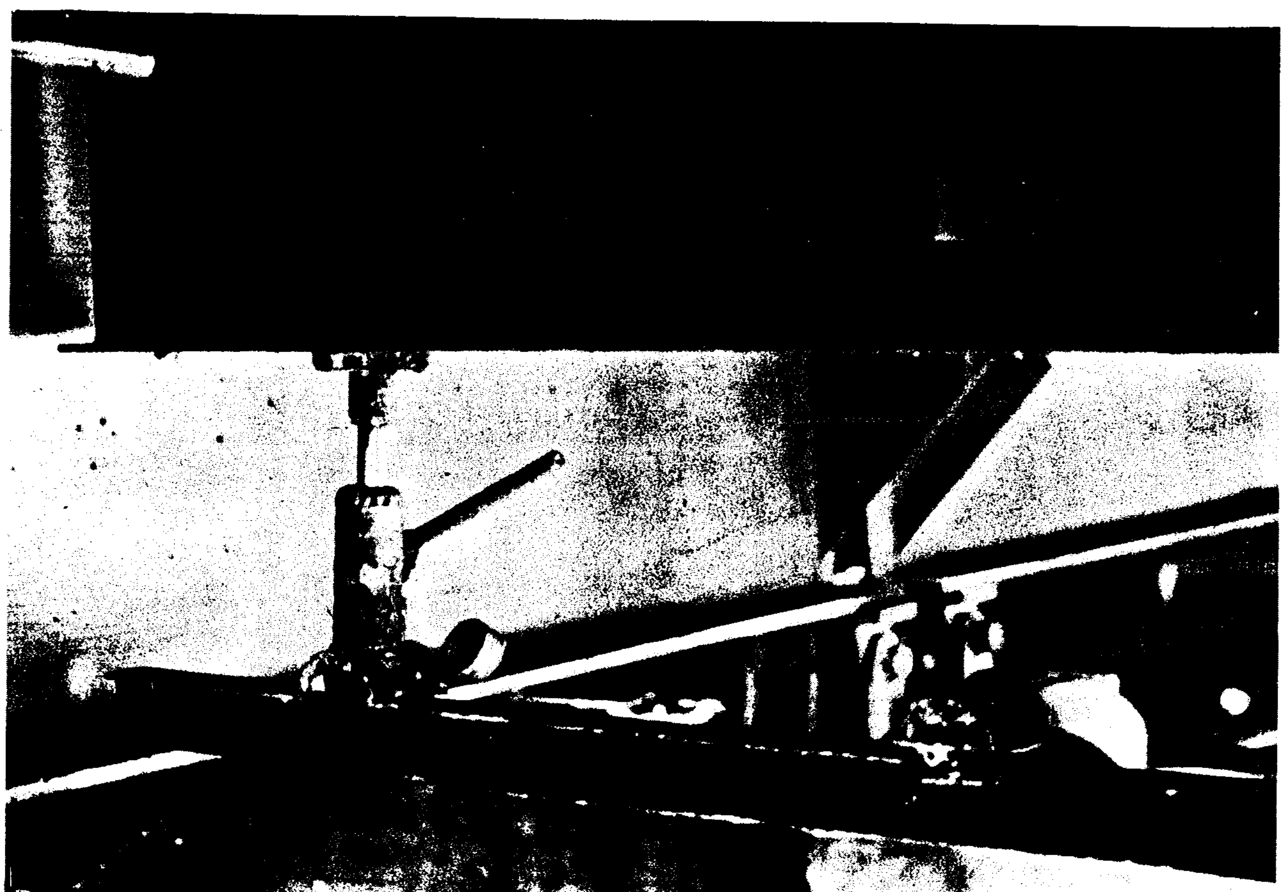


Figure 4.3. Jacking Equipment

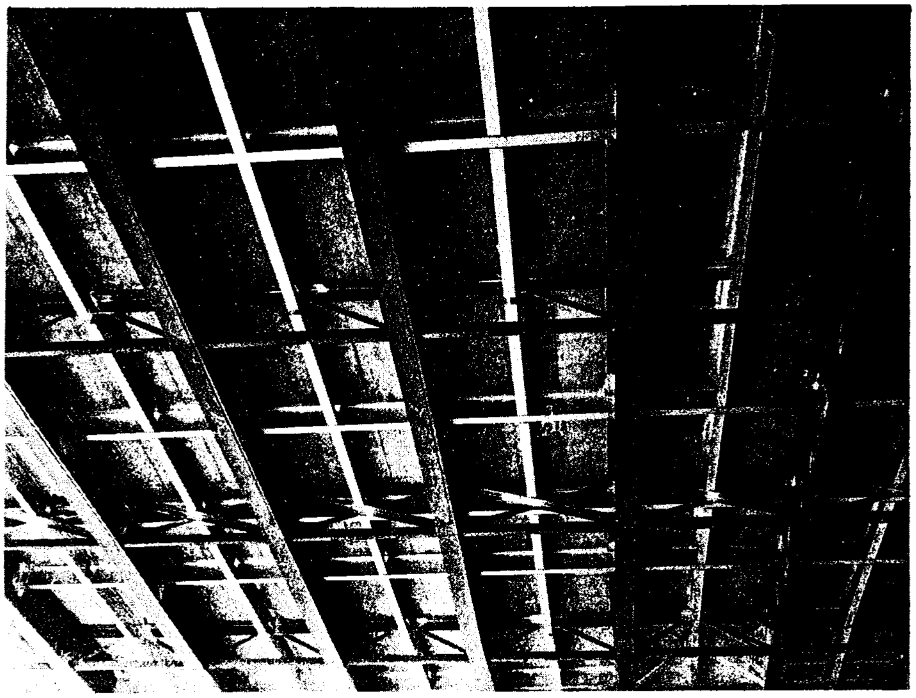


Figure 4.4. Strengthening System in Place

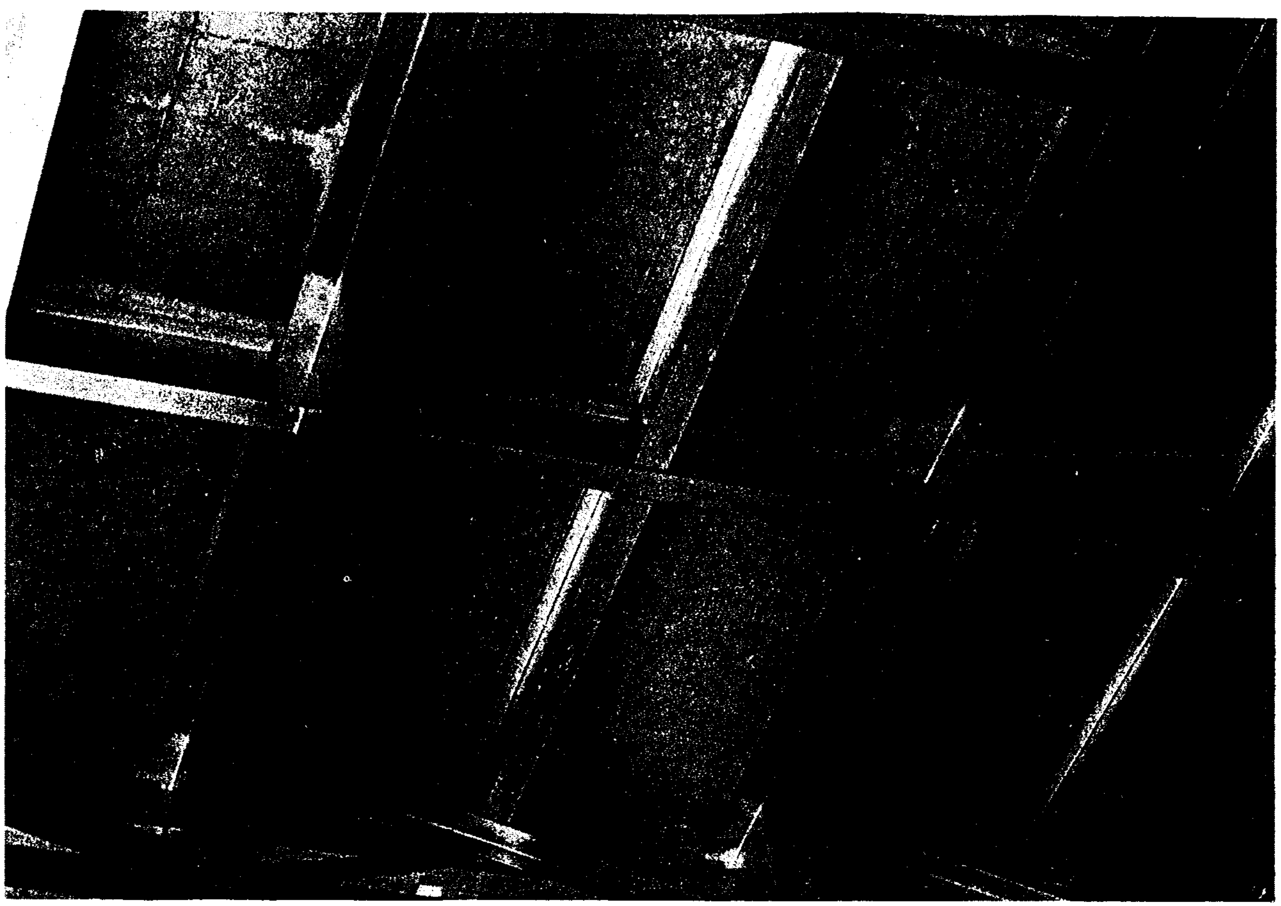


Figure 4.5. Typical Grid Beam Installation

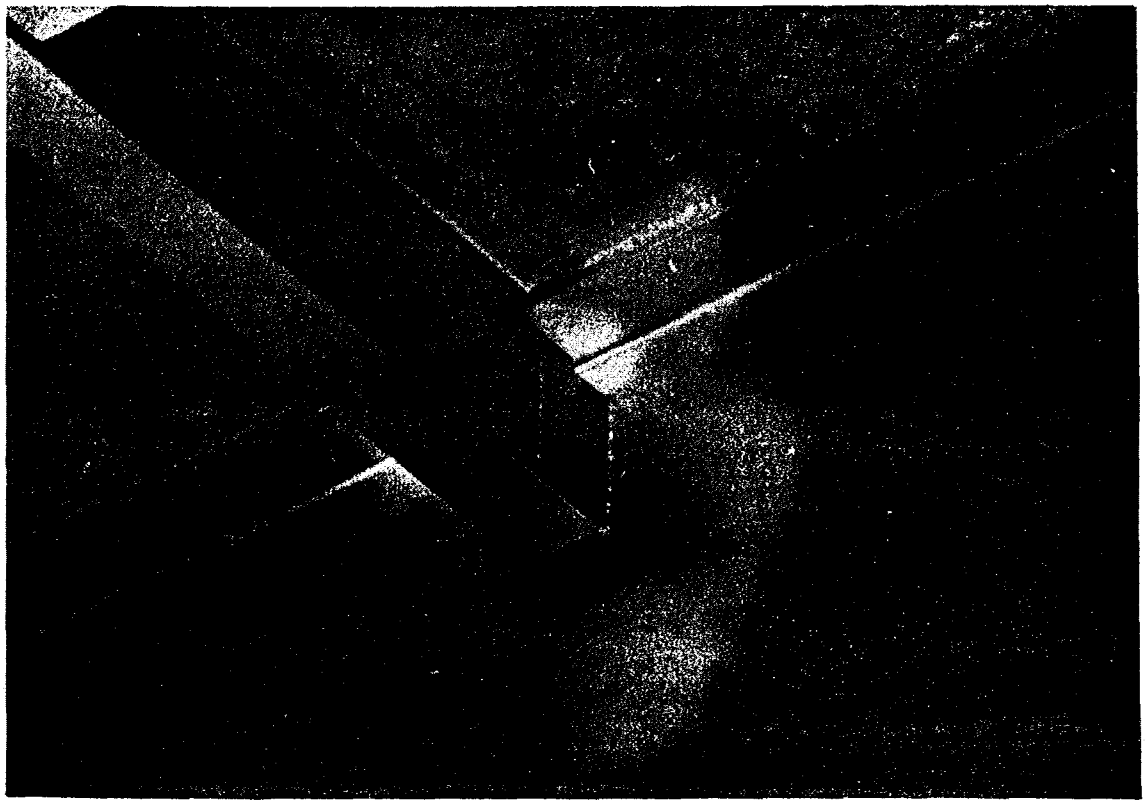


Figure 4.6. Typical Beam Connection

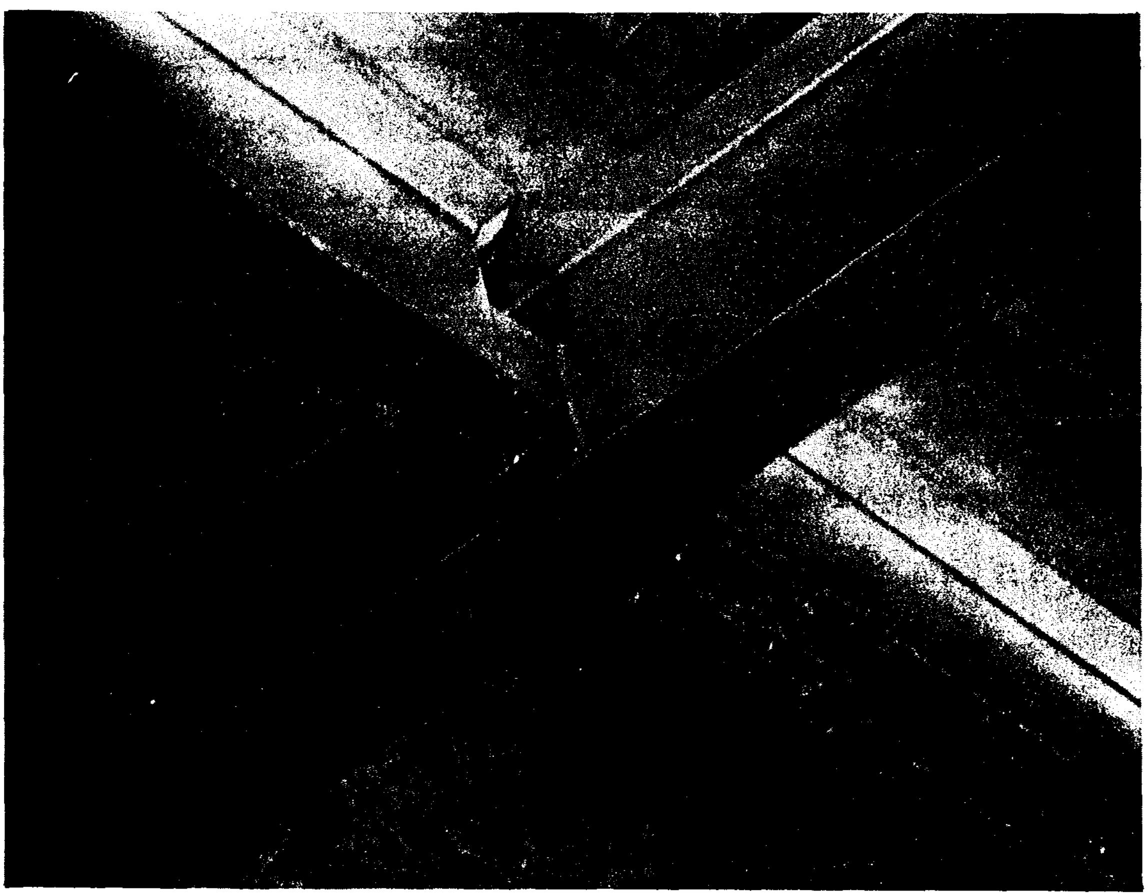


Figure 4.7. Typical Stringer - Floor beam Connection

CHAPTER V

STATIC LOAD TESTS OF SLAB STRENGTHENING SYSTEM

Static load tests were made to assist in evaluating the effectiveness of the slab strengthening system. Load tests were made on four panels, two in each structure, prior to and after installation of the strengthening system. The panels were selected as representative of the strengthened area and were located in the outside lanes so that testing could be conducted with minimum interference to traffic.

Loads

A loaded dump truck supplied the static load used in the test. The rear wheels, previously weighed on a commercial scale, were rolled to the centerline of a 96 inch x 70 inch (243.8 cm x 177.8 cm) steel plate which was centered on the test panel. The load, approximately 24,000 pounds (10,886 kg), was transferred to the concrete deck through a 29 inch (73.7 cm) diameter steel plate. Details of the load-plate are shown in Figure 5.7 and load plate positions are shown in Figures 5.3 and 5.4. Two additional load positions were used on Structure 98 after the strengthening beams were placed. Those two positions are shown in Figure 5.8.

The load was applied and removed three times in each test, except that four applications were made at position 4, Structure 97 because time permitted it. All gages were read before and after each loading.

Gages

Dial gages reading to 0.0001 in. (.00025 cm) were used to take deflection readings. These gages were mounted on frames, Figure 5.6, in positions shown in Figures 5.3 and 5.4. Electrical resistance gages were installed on the mid-panel transverse beam, top and bottom flanges as shown in Figure 5.5. Dummy gages were mounted on a small steel block which was set beside the active gage for temperature compensation.

Deck slab deflections were measured with the dial gages both before and after strengthening of the bridge. Strains were measured only on one of the strengthening beams, so there were no strain readings before strengthening.

Tests

The outside traffic lane was blocked off from traffic but other lanes were open. Truck and car traffic continued to use the open lanes throughout the test.

All gages were initially zeroed under no-load condition. The test vehicle was then run upon the load plate and gages were read. When the load was removed they were again read, and so on through three load applications (four applications in the instance noted earlier).

Under this procedure the test ran smoothly with no particular difficulties noted.

<u>Results of Load Tests</u>

Dial gage data were reduced to give deflection when the load was applied, and rebound when the load was removed. Strain gage data were reduced to give strain with each load application and each load removal. Gage readings are shown in Tables 5.1 and 5.2; deflections and strains are shown in Table 5.3.

Strains are plotted in Figure 5.9. The average deflections and rebounds are plotted in Figures 5.10 through 5.13. The strain gage readings show that there was very little horizontal shear transfer between slab and beams in Structure 97, whereas considerable interaction is indicated in Structure 98.

The deflections shown in Figures 5.10 through 5.13 clearly show a great stiffening effect of the added beams. The individual deflections shown in tables show some scatter, but they are generally in good agreement. In Table 5.3, load position 98-1-A, gage 4 before strengthening, the value shown for deflection at second loading is one of the values so far out of line that it is very likely incorrect. This is probably due to an incorrect reading of the gage. Other readings show that deflections sometimes increase and sometimes decrease as the load applications increase. These are very likely due, in part, at least, to the cracks in the deck slab that existed before the tests began.

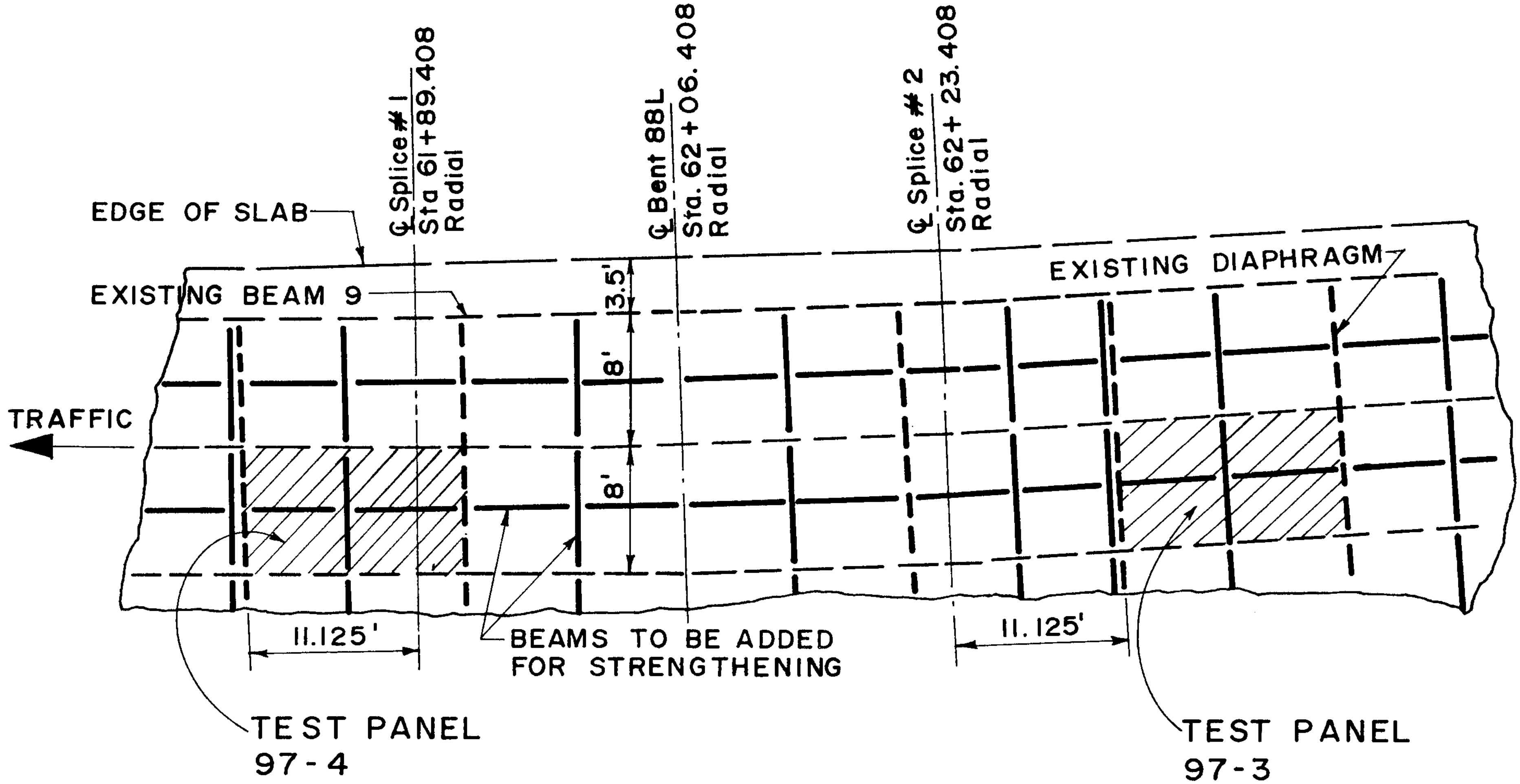


FIG. 5.1. STRUCTURE 97 -- LOCATIONS OF TEST PANELS.

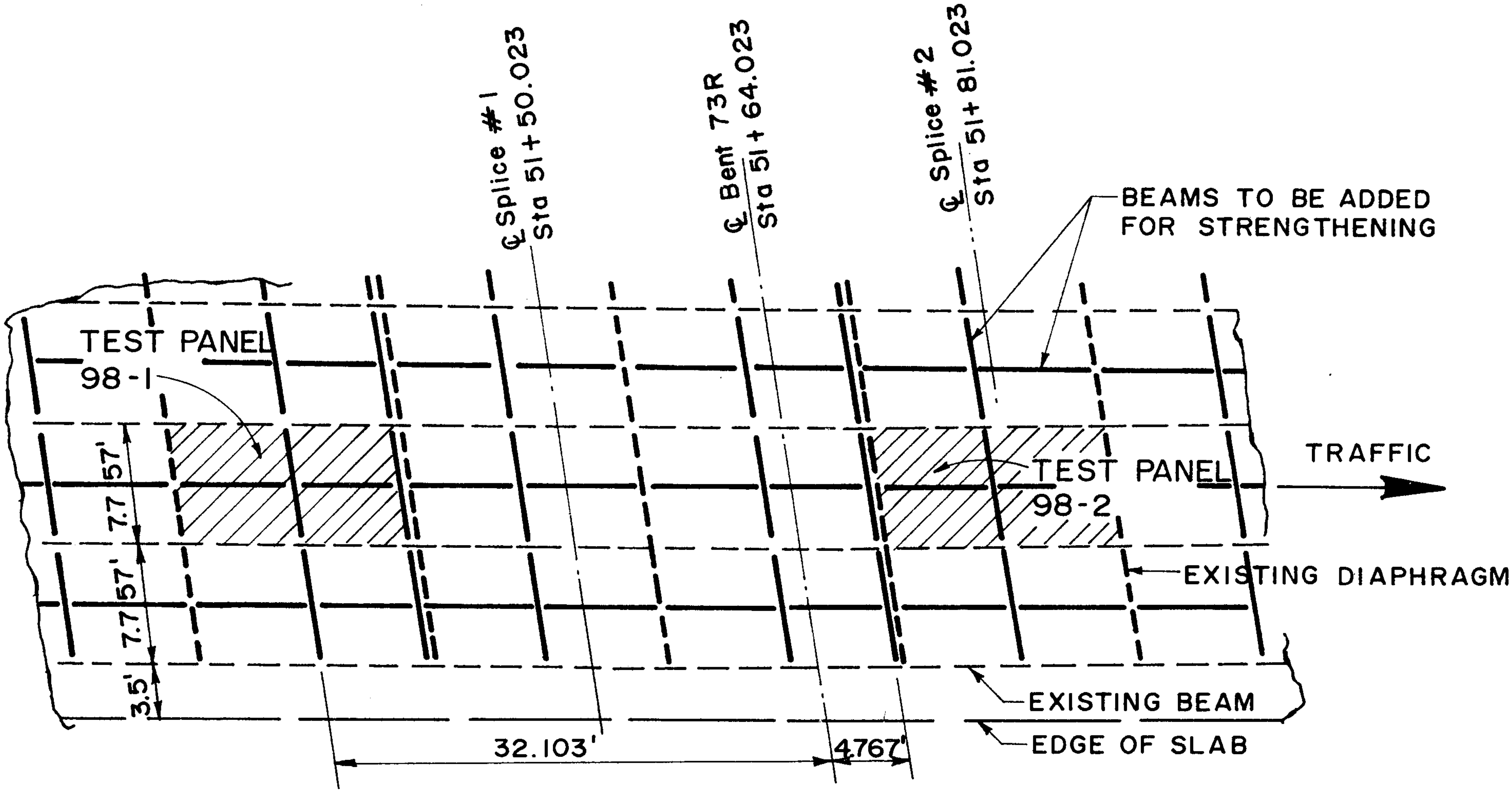


FIG. 5.2. STRUCTURE 98--LOCATIONS OF TEST PANELS.

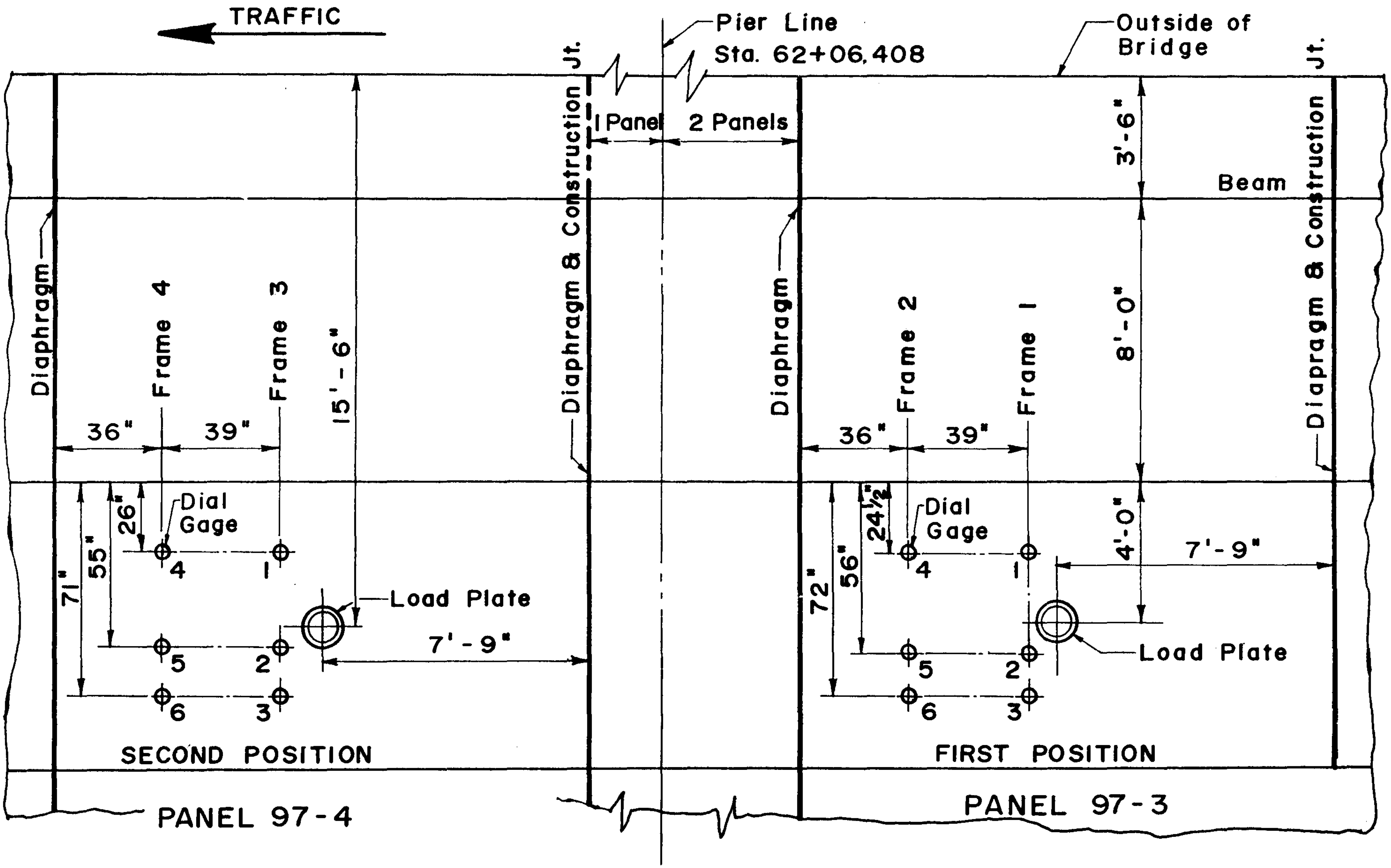


FIG. 5.3. POSITIONS OF DIAL GAGES AND LOAD PLATE-STRUCTURE 97.

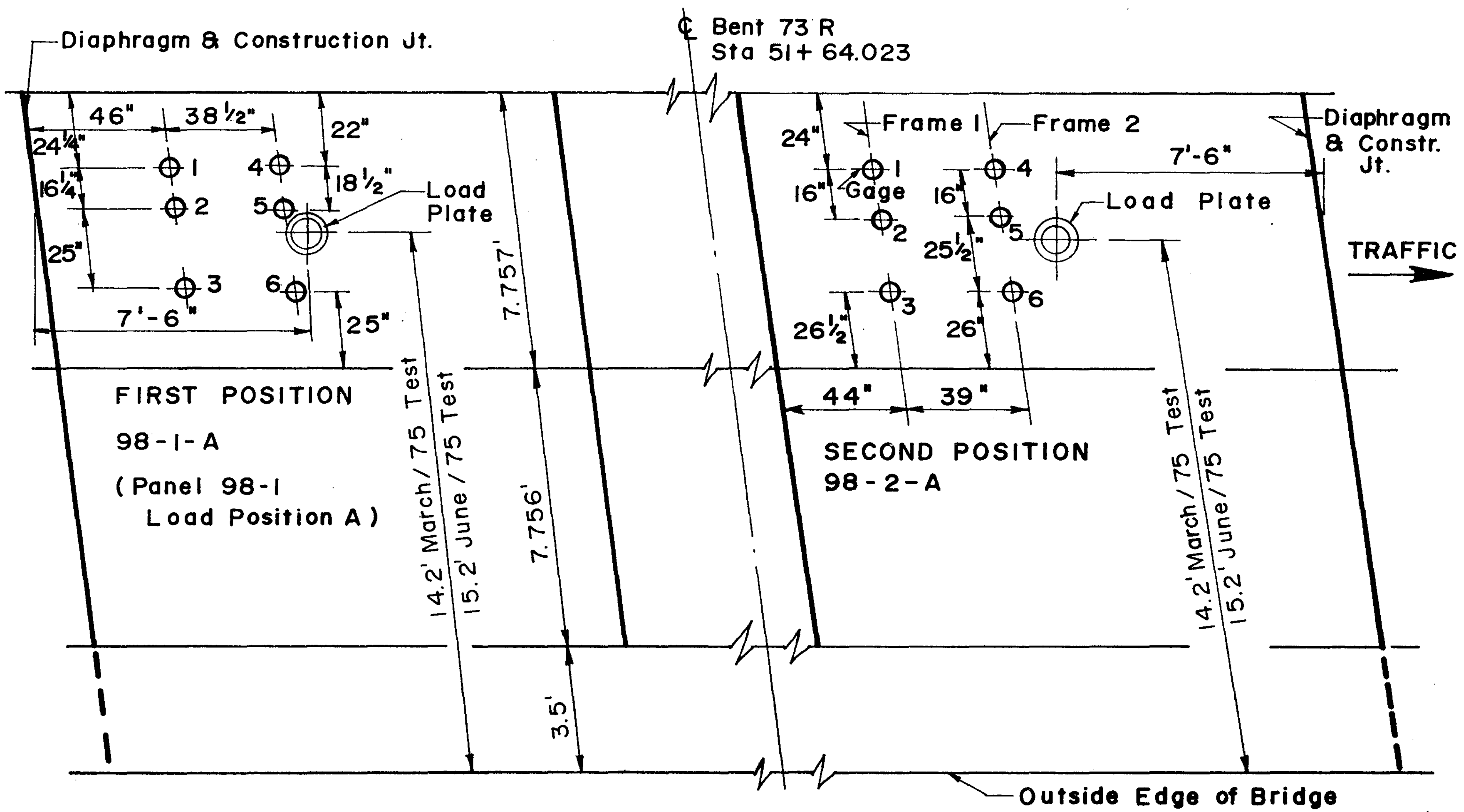
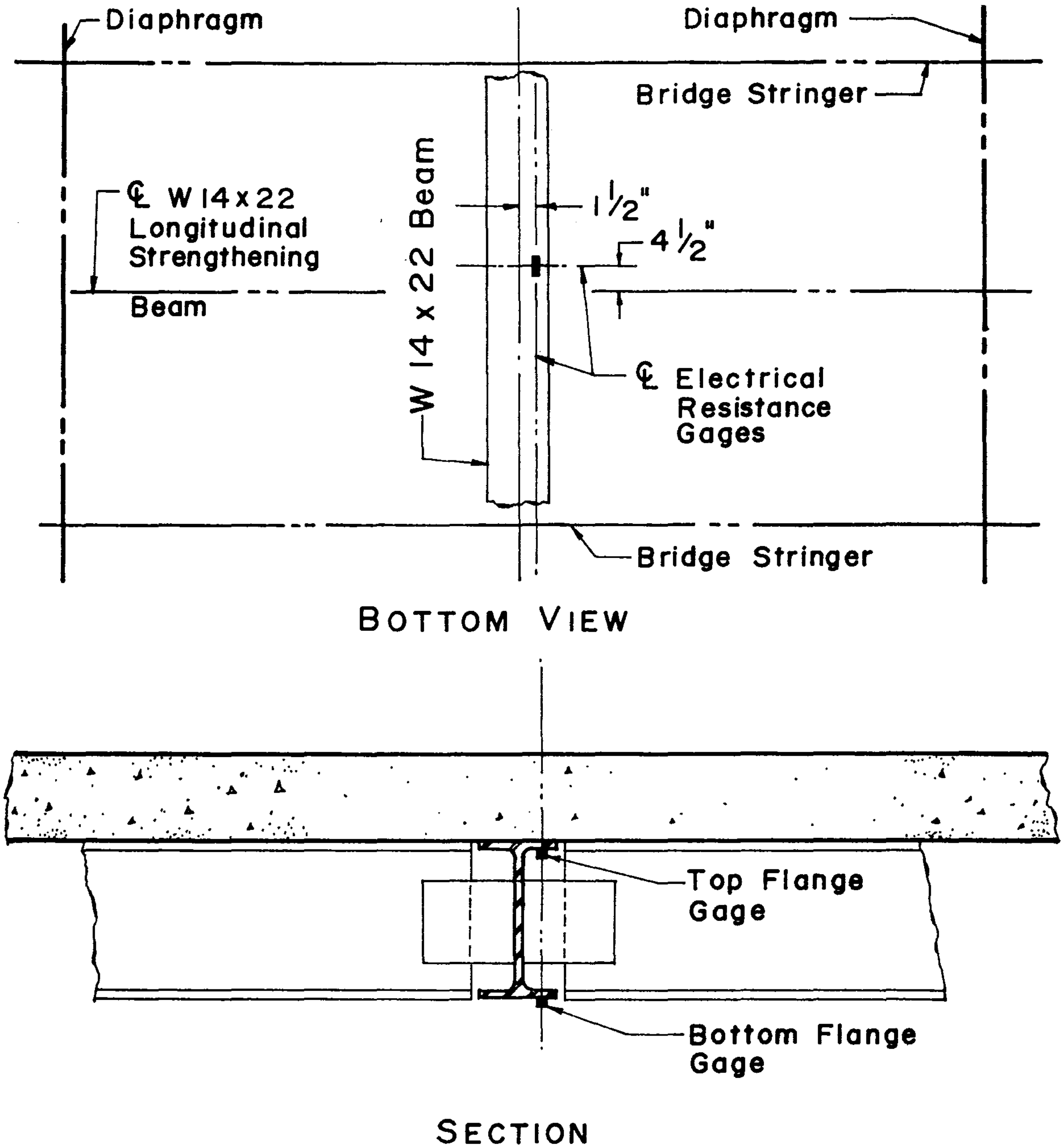


FIG. 5.4 POSITIONS OF DIAL GAGES AND LOAD PLATES - STRUCTURE 98.



Electrical Resistance Gages: Two single gages per location after strengthening only; Ailtech Weldable, Type SG-189, 120 ohm.

FIG. 5.5. POSITIONS OF ELECTRICAL GAGES.

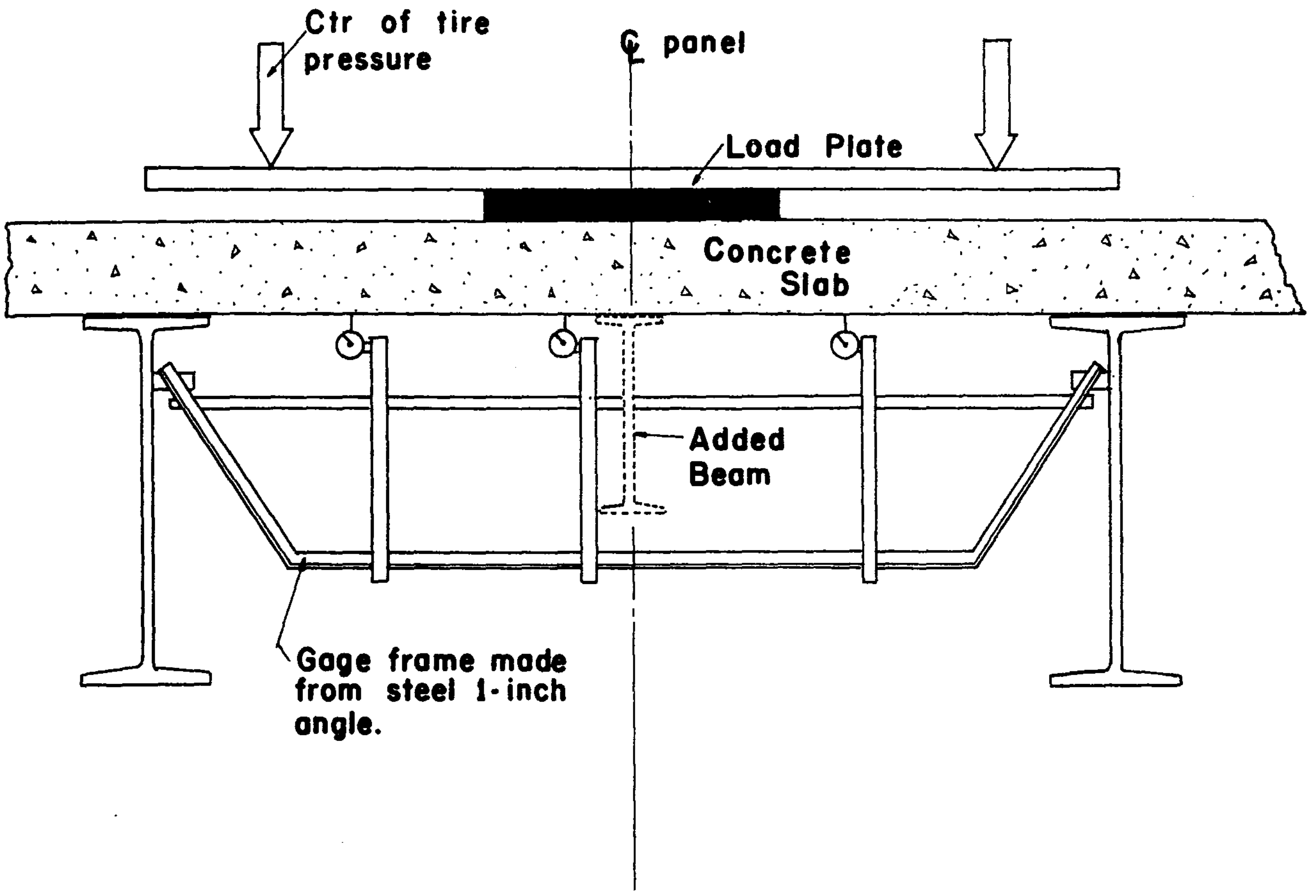


FIG. 5.6. GAGE MOUNT FOR DIAL GAGES.

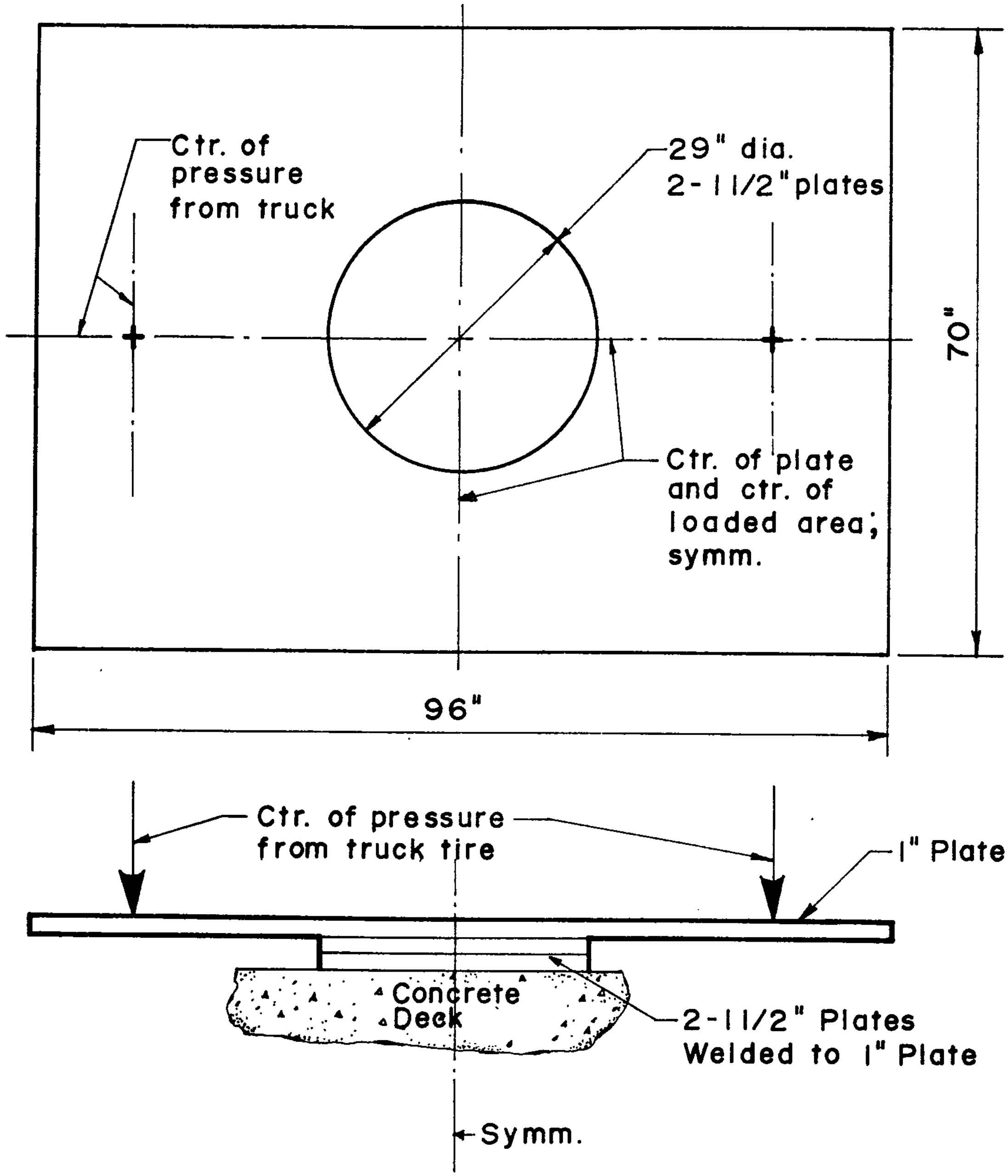


FIG. 5.7. LOAD-PLATE DETAILS.

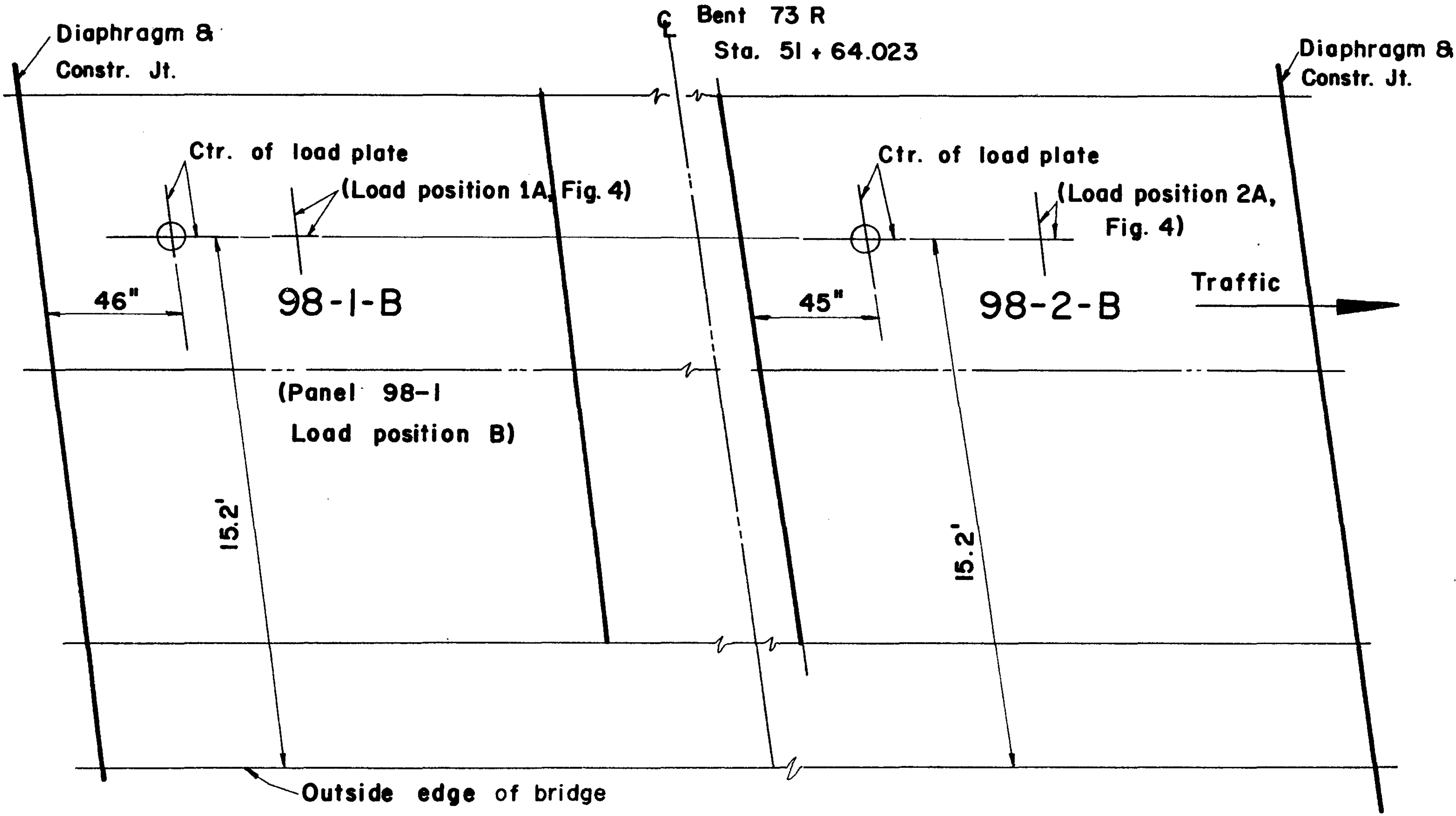


FIG. 5.8. STRUCTURE 98, ADDITIONAL LOAD POSITIONS, JUNE/75 TEST

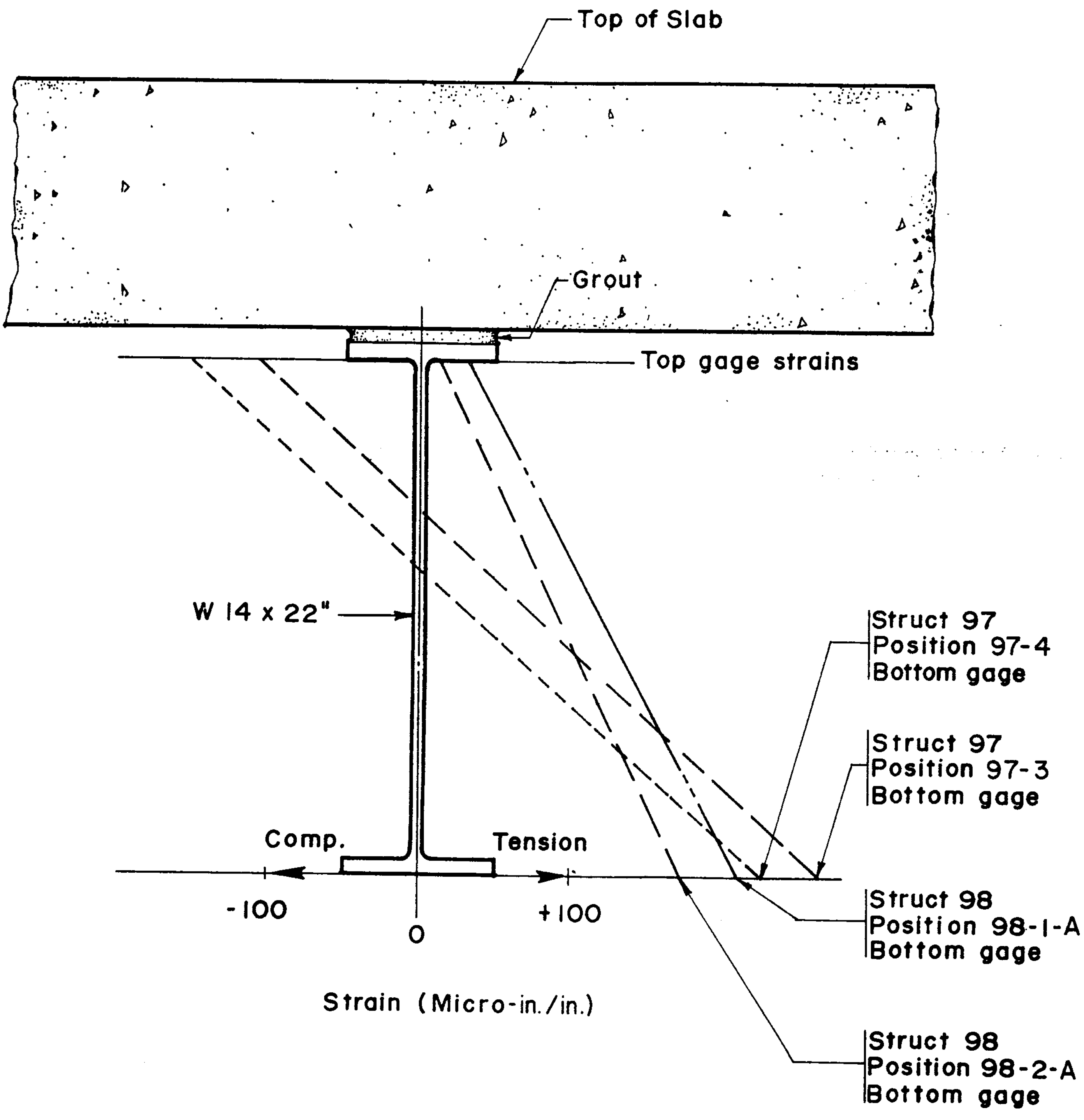


FIG. 5.9. STRAINS IN TRANSVERSE BEAM AT MIDSPAN.

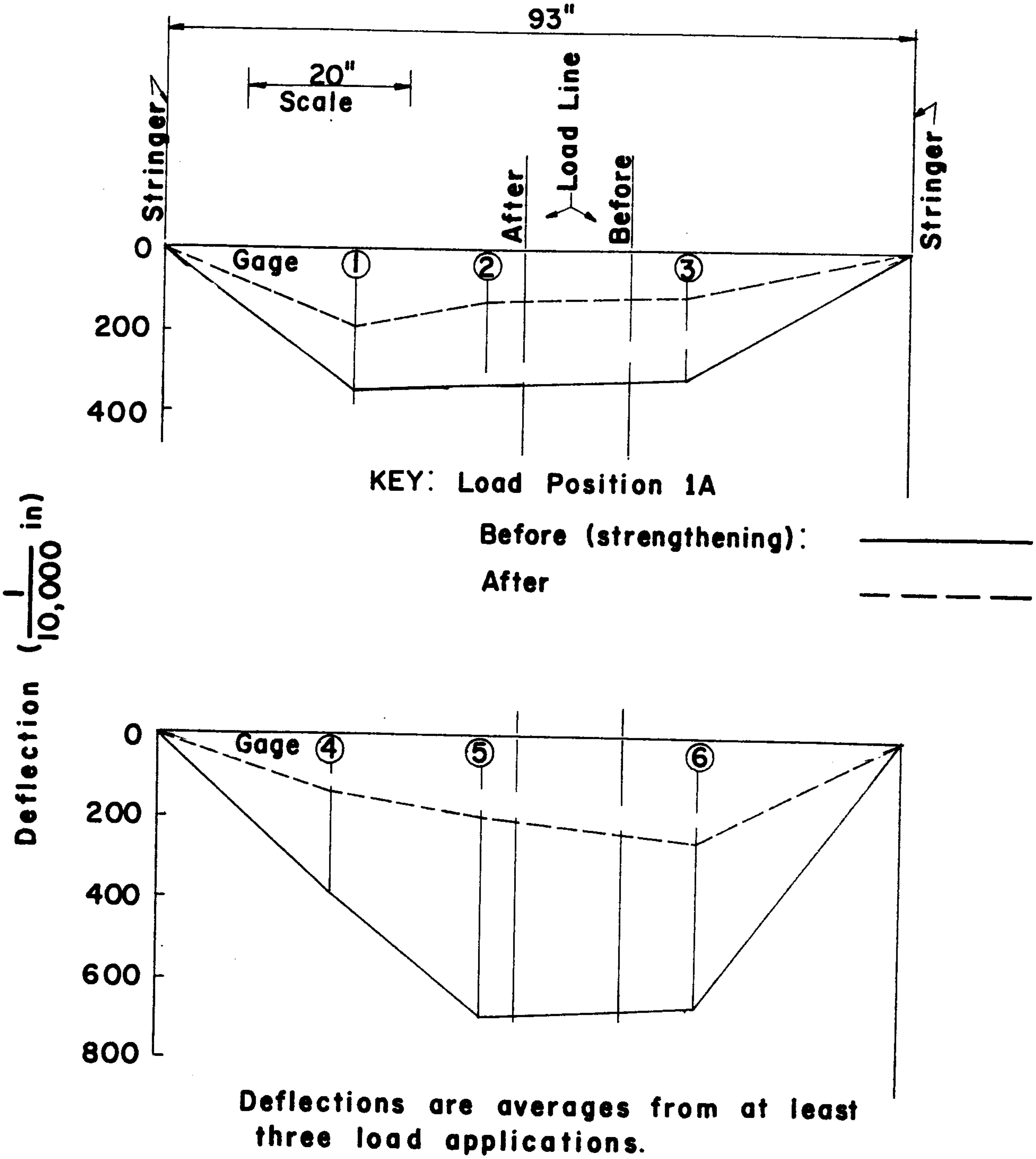
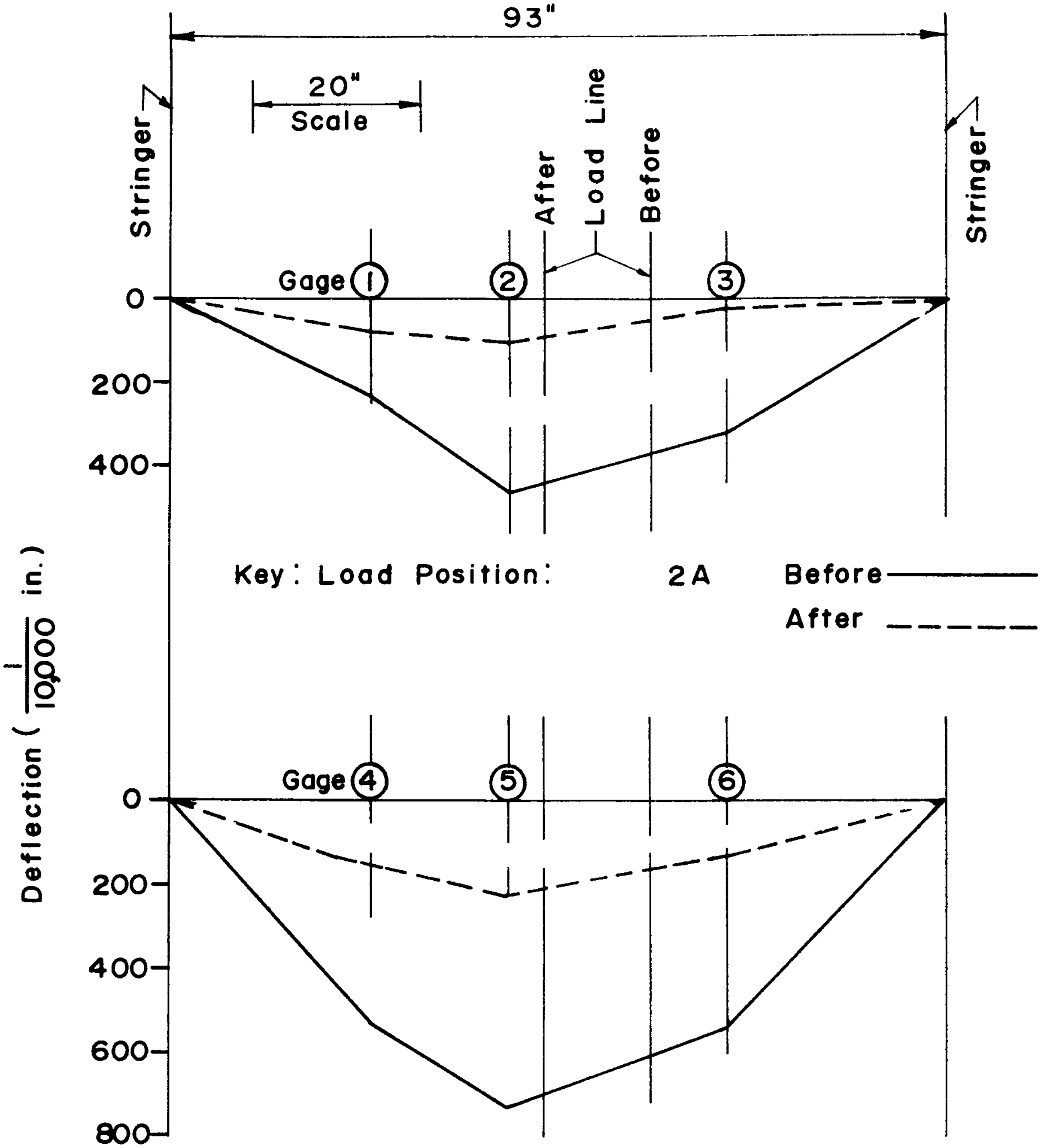


FIG. 5.10. SLAB DEFLECTIONS, PANEL 98-1-A



Deflections are averages from at least three load applications.

FIG. 5.11. SLAB DEFLECTIONS, PANEL 98-2-A

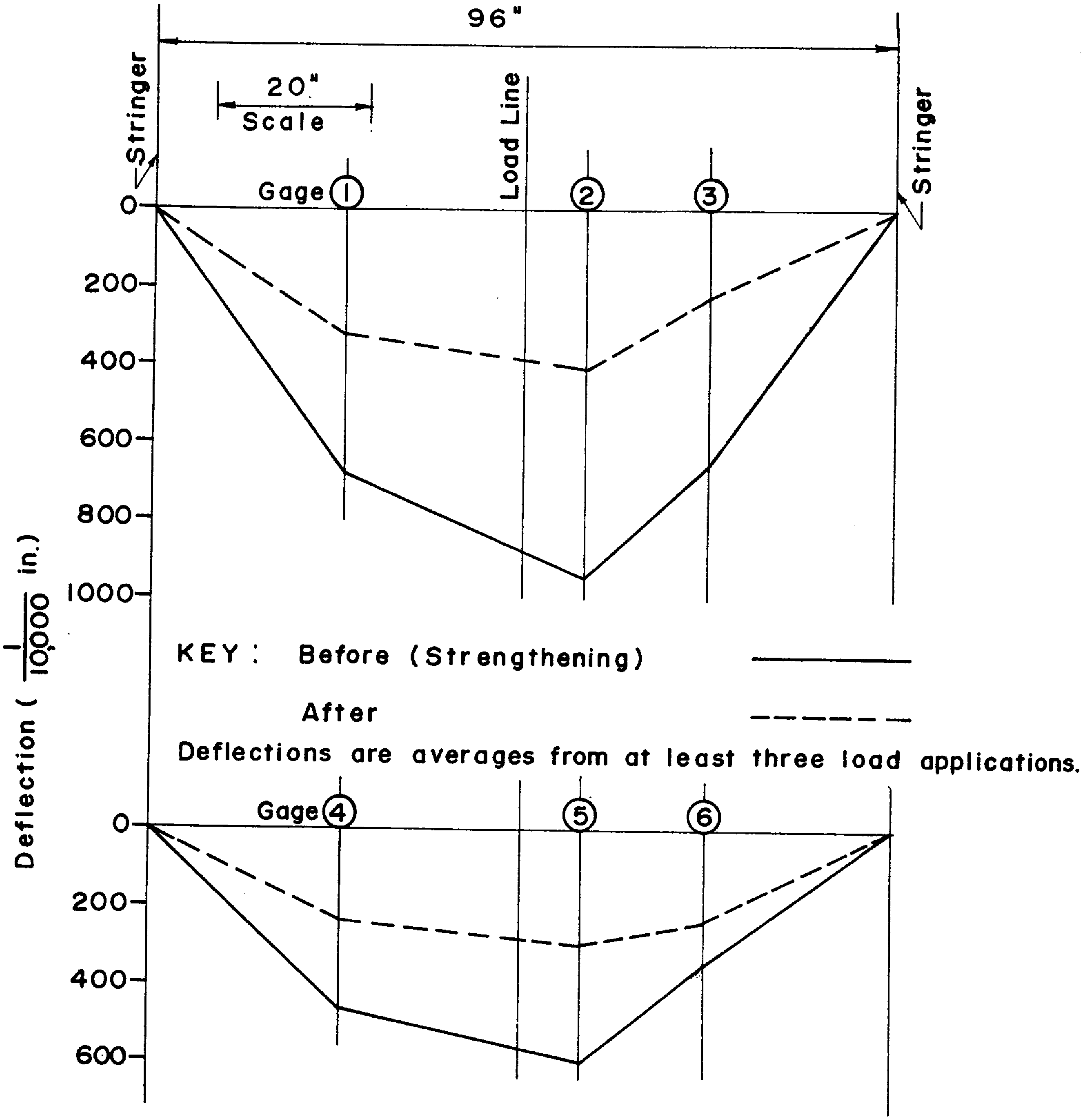


FIG. 5.12. SLAB DEFLECTION, PANEL 97-3.

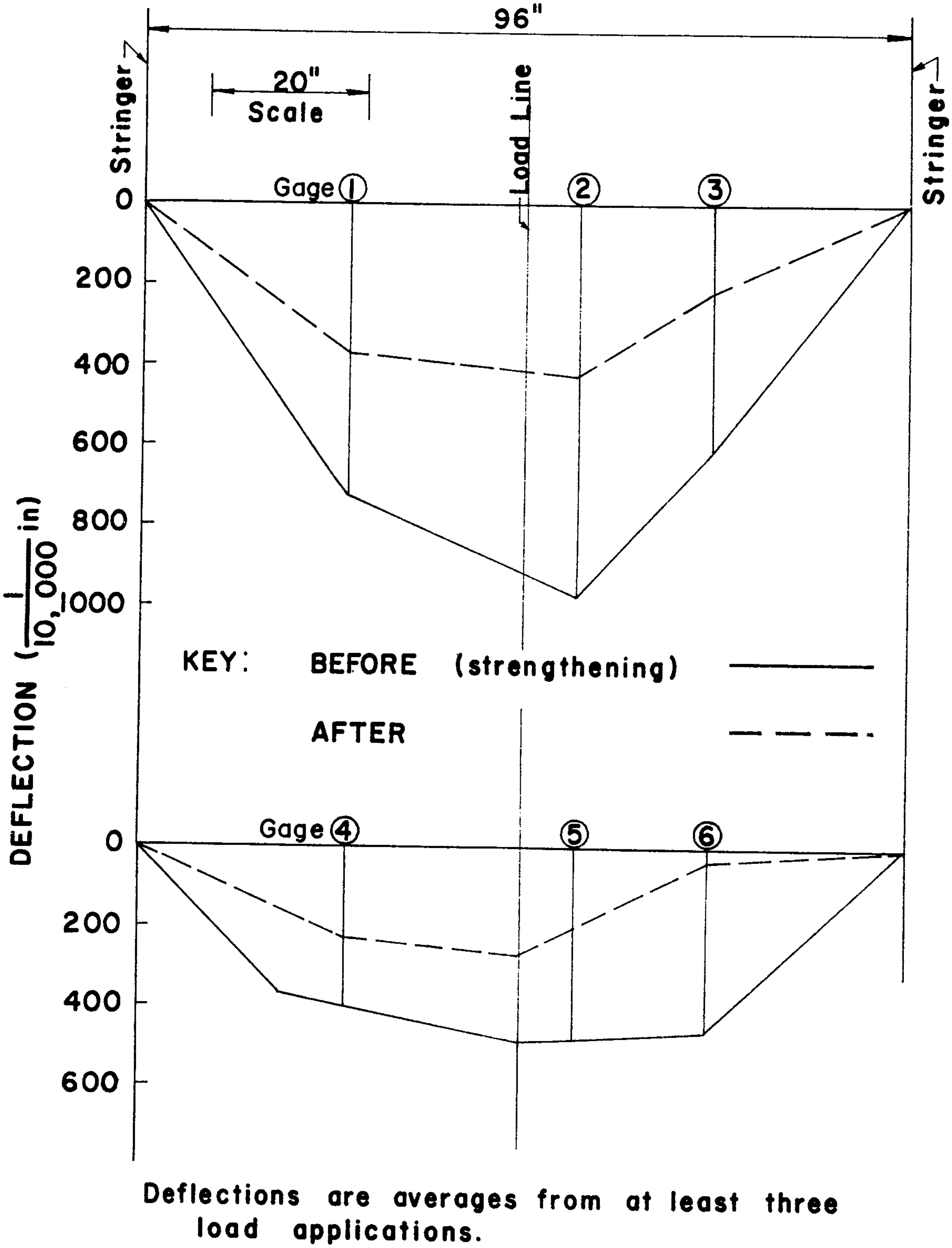
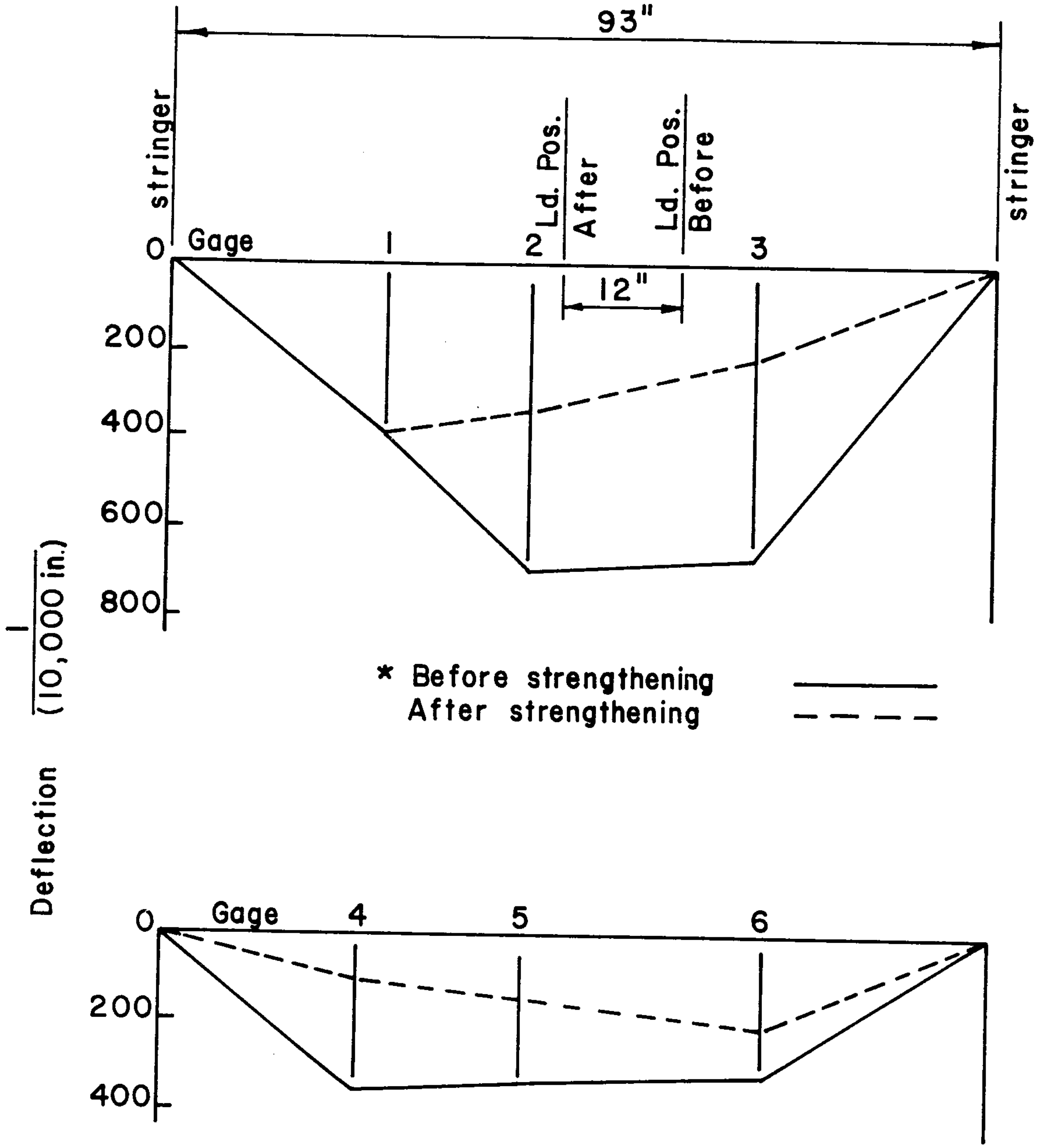
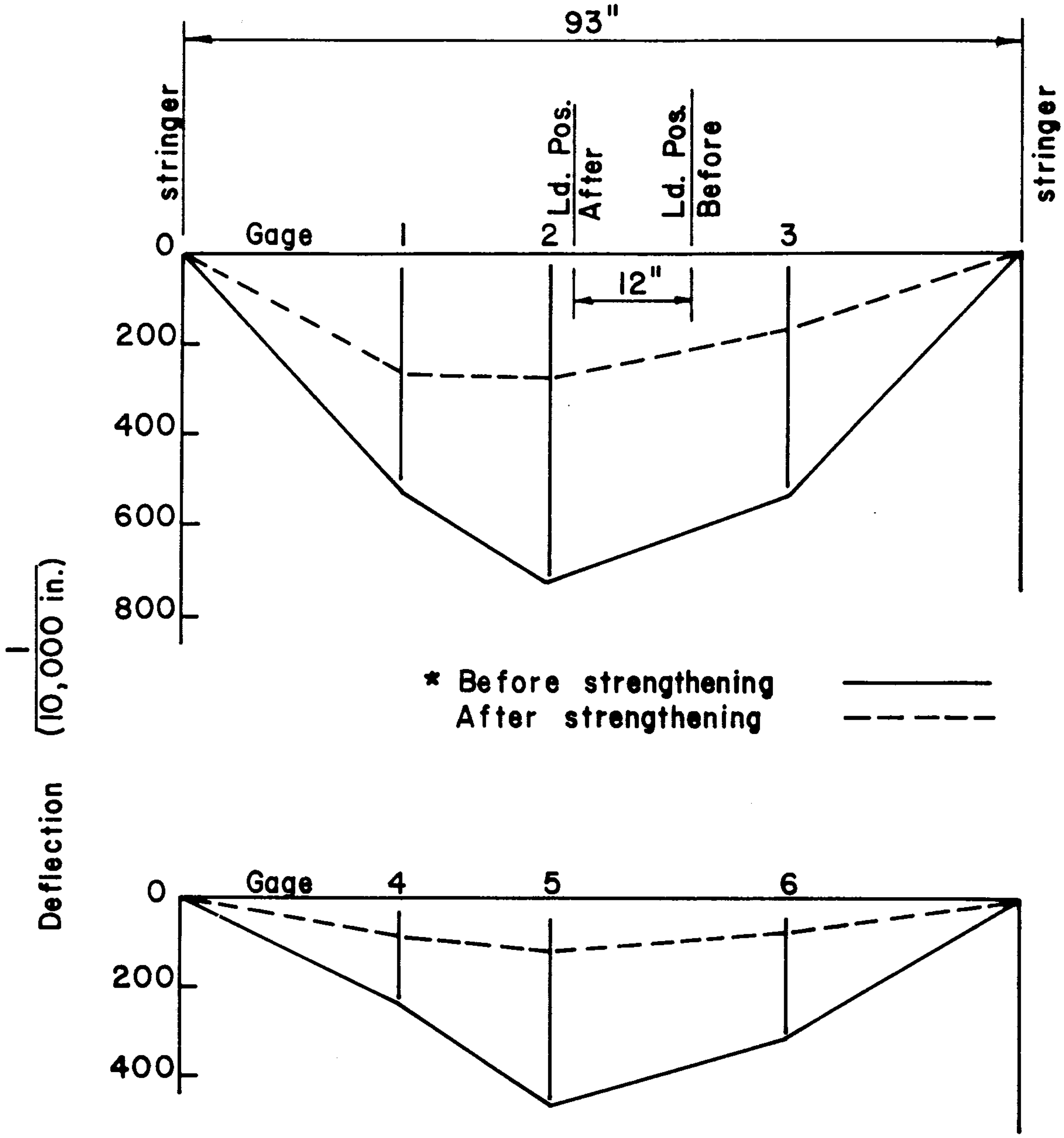


FIG. 5.13. SLAB DEFLECTIONS, PANEL 97-4.



* Before strengthening deflections are assumed and are based on measurements made at load position 98-1-A

FIG. 5.14. SLAB DEFLECTIONS PANEL 98-1-B



* Before strengthening deflections are assumed and are based on measurements made at load position 98-2-A

FIG. 5.15. SLAB DEFLECTIONS PANEL 98-2-B

TABLE 5.1 GAGE READINGS BEFORE STRENGTHENING

Struct.	Position	Load	GAGE NUMBER 1	2	3	4	5	6
			(1/10,000 inch)					
98	1A	zero	800	800	700	700	100	300
		24.2^k	155	253	012	236	850	945
		zero	803	675	600	790	090	045
		24.2	158	905	905	730	815	650
		zero	805	670	610	380	310	922
		24.2	160	925	923	705	025	510
		zero	810	670	610	375	305	920
	2A	zero	900	700	500	900	500	500
		24.2	143	183	830	459	346	090
		zero	915	720	510	922	638	535
		24.2	141	177	815	460	345	100
		zero	920	723	527	931	649	540
		24.2	148	187	831	465	356	100
		zero	922	728	515	937	656	545
97	3	zero	400	700	100	100	000	500
		24.14	132	690	755	511	640	865
		zero	432	740	117	117	005	503
		24.14	133	675	750	591	610	850
		zero	429	740	115	111	005	500
		24.14	111	700	785	575	605	855
		zero	432	745	125	112	005	505
	4	zero	200	700	900	000	500	900
		24.14	914	723	658	377	010	380
		zero	203	727	904	013	510	905
		24.14	956	714	530	400	020	368
		zero	216	737	915	010	520	911
		24.14	942	717	537	384	005	358
		zero	213	742	915	011	515	907

TABLE 5.2 GAGE READINGS AFTER STRENGTHENING

Struct.	Position	Load	GAGE NUMBER 1	2	3	4	5	6	Electrical Top	Electrical Bottom
			(1/10,000 inch)							
98	1B	zero	2200	3400	1600	1700	3400	0400		
		23.66k	110	3738	2830	1805	3551	0620		
		zero	2210	3415	1610	1702	3405	0410		
		23.66	113	3710	2840	1805	3523	0615		
		zero	2213	3385	1610	1705	3375	0400		
		23.66	105	3715	2840	1817	3530	0620		
		zero	2215	3385	1620	1700	3370	0405		
	1A	zero	2200	3355	1690	1705	3400	0410	+330	+627
		23.66k	2384	3474	1793	1845	3698	0670	364	842
		zero	2195	3347	1685	1708	3404	0413	335	638
		23.66	2365	3447	1780	1825	3570	0675	361	847
		zero	2287	3336	1680	1707	3393	0415	330	640
		23.66	2380	3460	1790	1845	3570	0670	360	848
		zero	2185	3333	1675	1705	3378	0413	332	635
	2B	zero	331	2538	2510	4189	2690	1725		
		23.66	605	2815	2656	4277	2791	1800		
		zero	335	2542	2500	4187	2675	1720		
		23.66	600	2818	2660	4268	2784	1800		
		zero	338	2543	2500	4178	2662	1715		
		23.66	605	2845	2693	4267	2773	1794		
		zero	348	2580	2503	4172	2550	1710		
	2A	zero	325	2558	2590	4175	2555	1716	665	855
		23.66	395	2661	2564	4299	2778	1834	673	1028
		zero	322	2557	2585	4167	2510	1700	664	858
		23.66	397	2662	2563	4299	2782	1835	670	1029
		zero	320	2558	2585	4161	2581	1700	657	857
		23.66	398	2665	2565	4295	2659	1831	668	1030
97	3	zero	1600	1600	800	3100	4400	1200	-100	-130
		23.66k	1945	2025	1026	3353	4613	1448	-213	+135
		zero	1610	1602	803	3110	4408	1205	-104	-128
		23.66	1940	2030	1031	3360	4620	1453	-218	+135
		zero	1610	1603	802	3110	4407	1202	-110	-128
		23.66	1930	2021	1027	3340	4697	1436	-218	+135
		zero		Not taken						
	4	zero	3700	2600	1700	0200	1000	0800	-200	-665
		23.66	4077	3040	1927	0432	1387	0930	-350	-440
		zero	3605	2595	1708	0200	1007	0898	-199	-657
		23.66	3960	2998	1916	0420	1370	908	-350	-432
		zero	3610	2600	1704	0200	1005	895	-210	-660
		23.66	3990	3040	1920	0450	1307	935	-363	-422
		zero	3610	2602	1710	0210	1009	897	-210	-653
		23.66	3977	3032	1933	0435	1292	922	-360	-432
		zero	3610	2603	1715	0200	1015	892	-206	-650

TABLE 5.3 DEFLECTIONS AND STRAINS

Struct.	Load Position	Load	Deflection ($\frac{1}{10000}$ in.) at Gage Number 1	2	3	4	5	6	Strain Top	(microin) Bottom
		(24.2 kip) → Before strengthening								
98	1A	Load	355 down	453	312	536	750	645		
		Unload	352 up	578	412	446	760	900		
		Load	355	230	305	out 960	725	605		
		Unload	353	235	295	350	505	728		
		Load	355	255	313	325	715	588		
		Unload	350	255	313	330	720	590		
		Avg.	(353)	(334)	(325)	(397)	(696)	(676)		
	2A	Load	243 down	483	330	559	846	590		
		Unload	228 up	463	320	537	708	555		
		Load	226	457	305	462	707	565		
		Unload	221	454	288	529	696	560		
		Load	228	464	304	534	707	560		
		Unload	226	459	316	528	700	555		
		Avg.	(229)	(463)	(310)	(525)	(727)	(564)		
		23.66^{k} → After strengthening								
	1A	Load	184 down	119	103	140	198	260	+ 34	+215
		Unload	189 up	127	108	137	194	257	- 29	-204
		Load	170	100	95	117	166	262	+ 26	+209
		Unload	178	111	100	118	177	260	- 31	-207
		Load	193	124	110	138	177	255	+ 30	+208
		Unload	195	127	115	140	192	257	- 28	-213
		Avg.	(185)	(118)	(105)	(132)	(184)	(258)	+(30)	+(209)
	1B	Load	410 down	338	230	105	151	220		
		Unload	400 up	323	200	103	146	210		
		Load	403	295	230	103	118	205		
		Unload	400	325	230	100	148	215		
		Load	392	330	230	112	155	220		
		Unload	390	330	220	117	160	215		
		Avg.	(399)	(323)	(227)	(107)	(146)	(214)		

TABLE 5.3 DEFLECTIONS AND STRAINS (continued)

Struct.	Position	Load	Deflection $\left(\frac{1}{10000}\text{ in.}\right)$ at Gage Number 1	2	3	4	5	6	Strain (microin) Top	Bottom
	2A	Load	70 down	103	26	124	223	118	+ 8	+173
		Unload	73 up	104	21	132	268	134	- 9	-170
		Load	75	105	22	132	272	135	+ 6	+171
		Unload	77	104	22	138	201	135	- 13	-172
		Load	78	107	20	134	178	131	+ 11	+173
		Avg.	(75)	(105)	(22)	(132)	(228)	(131)	(+9)	(+172)
	2B	Load	274 down	277	146	88	101	75		
		Unload	270 up	273	156	90	116	80		
		Load	265	276	160	81	111	80		
		Unload	262	275	160	90	122	85		
		Load	267	302	193	89	111	81		
		Unload	257	265	190	95	123	84		
		Avg.	(266)	(278)	(168)	(89)	(114)	(81)		
		24.14^k	Before strengthening							
97	3	Load	732 down	990	655	411	640	365		
		Unload	700 up	950	638	394	635	362		
		Load	701	935	633	474	605	347		
		Unload	704	935	635	480	605	350		
		Load	682	960	670	464	600	355		
		Unload	679	955	660	463	600	350		
		Avg.	(700)	(954)	(649)	(448)	(614)	(355)		
	4	Load	714 down	1023	758	377	510	480		
		Unload	711 up	996	754	364	500	475		
		Load	753	987	626	387	510	463		
		Unload	740	977	615	390	500	457		
		Load	726	980	622	374	485	447		
		Unload	729	975	622	373	490	451		
		Avg.	(729)	(990)	(666)	(378)	(499)	(462)		

TABLE 5.3 DEFLECTIONS AND STRAINS (continued)

Struct.	Load Position	Load	Deflection $\left(\frac{1}{10000}\text{ in.}\right)$ at Gage Number 1	2	3	4	5	6	Strain (microin) Top	Bottom
		23.66^k	After strengthening							
97	3	Load	345 down	425	226	253	213	248	-113	+265
		Unload	335 up	423	223	243	205	243	+109	-263
		Load	330	428	228	250	212	248	-114	+263
		Unload	330	427	229	250	213	251	+108	-263
		Load	320	418	225	230	290	234	-108	+263
		Unload	Not recorded							
		Avg.	(332)	(424)	(226)	(245)	(227)	(245)	(-110)	(+263)
	4	Load	377 down	440	227	232	387	130	-150	+225
		Unload	472 up	445	219	232	380	32	+151	-217
		Load	355	403	208	220	363	10	-151	+225
		Unload	350	398	212	220	365	13	+140	-228
		Load	380	440	216	250	202	40	-153	+238
		Unload	380	438	210	240	298	38	+153	-231
		Load	367	430	223	225	283	25	-150	+231
		Unload	367	429	218	235	277	30	+154	-218
		Avg.	(381)	(428)	(217)	(232)	(332)	(27)	(-150)	(+227)

TABLE 5.4 PERCENT REDUCTION OF SLAB DEFLECTIONS, STRUCTURE NO. 98

	Deflection, inches		
	Before Strengthening	After Strengthening	Reduction in Deflection (%)
Load Pos. 1-A			
Gage Pos. 1	.0353	.0185	47
2	.0334	.0118	64
3	.0325	.0105	67
4	.0397	.0132	66
5	.0696	.0184	73
6	.0676	.0258	62
			(avg) 63
Load Pos. 2-A			
Gage Pos. 1	.0229	.0075	67
2	.0463	.0105	77
3	.0310	.0022	92
4	.0525	.0132	74
5	.0727	.0228	68
6	.0564	.0131	76
			(avg) 75
Load Pos. 1-B*			
Gage Pos. 1	.0397	.0399	-
2	.0696	.0323	54
3	.0676	.0227	66
4	.0353	.0107	70
5	.0334	.0146	56
6	.0325	.0214	34
			(avg) 56
Load Pos. 2-B*			
Gage Pos. 1	.0525	.0266	49
2	.0727	.0278	62
3	.0564	.0168	70
4	.0229	.0089	61
5	.0463	.0114	75
6	.0310	.0081	74
			(avg) 65

* Deflections before strengthening at load positions 1-B and 2-B are assumed and are based on measurements made at load positions 1-A and 2-A.

TABLE 5.5 PERCENT REDUCTION OF SLAB DEFLECTIONS, STRUCTURE NO. 97

	Deflection, inches		
	Before Strengthening	After Strengthening	Reduction in Deflection (%)
Load Pos. 3			
Gage Pos. 1	.0700	.0332	53
2	.0954	.0424	56
3	.0649	.0226	65
4	.0448	.0245	45
5	.0614	.0227	63
6	.0355	.0245	31
			(avg) 52
Load Pos. 4			
Gage Pos. 1	.0729	.0381	47
2	.0990	.0428	57
3	.0666	.0217	67
4	.0378	.0232	39
5	.0499	.0332	33
6	.0462	.0026	91
			(avg) 56

CHAPTER VI

SUMMARY AND CONCLUSIONS

A method has been developed to strengthen existing deteriorating bridge slabs from underneath, thus eliminating the disruption of traffic. A small portion of two structures near downtown Houston, Texas, was strengthened using this method. The effectiveness of this strengthening was evaluated by means of field measurements of deflection and strain under static wheel loading before and after slab strengthening. Based on this evaluation, the following conclusions are made:

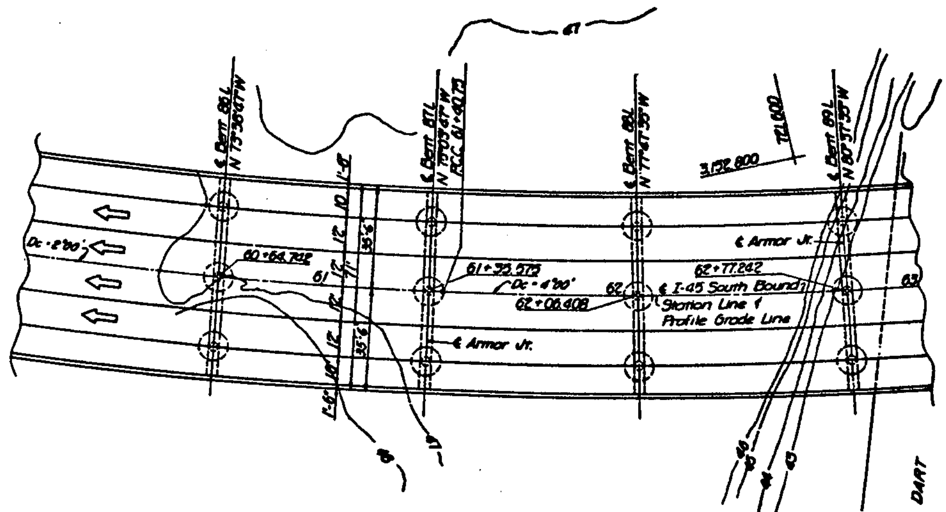
1. The added grid beam system reduced slab deflections by an average of 62% when loaded at midspan of a floorbeam and an average of 56% when loaded at midspan of a stringer. Calculated design values were 63% and 46% respectively.
2. Stresses in the bottom flange of the added floorbeams were calculated from measured strains and ranged from 5160 psi (35.6 MPa) to 7890 psi (54.4 MPa) with an average value of 6558 psi (45.2 MPa). This compares with a stress of 12,400 psi (85.5 MPa) calculated using the test load and design load distribution coefficients.

3. The strengthening system can be erected under live load conditions without a great deal of difficulty.
4. This strengthening system has not healed the existing slab cracking. Some type of surface sealing will be required to prevent further surface decomposition by exposure to contaminants.
5. Should the slab ultimately require replacement, the added beams will be beneficial to the new slab.
6. The cost of this project was high, 61% over the Engineer's estimate. This was probably due to the experimental nature of the project and the critical shortage of steel at the time of bidding and future costs should be lower.

Attempts at using aerial photographic techniques to evaluate the rate of slab deterioration have not been entirely successful. Photographs were made using an airplane equipped for photogrametry, and a helicopter. A series of photographs was also taken from a 60-foot (18.3 m) platform, however, none of these methods produced the detail necessary to evaluate the slab cracking patterns. The best results have been obtained from aerial photographs which show the areas of slab that have been patched. This gives a rough estimate of the current amount of deterioration and these can be compared with future photographs to obtain an estimate of the rate of deterioration.

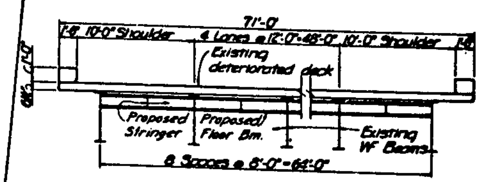
APPENDIX A

DETAILS OF SLAB STRENGTHENING SYSTEM



PARTIAL PLAN – EXISTING STRUCTURE NO. 97

I-45 SOUTHBOUND

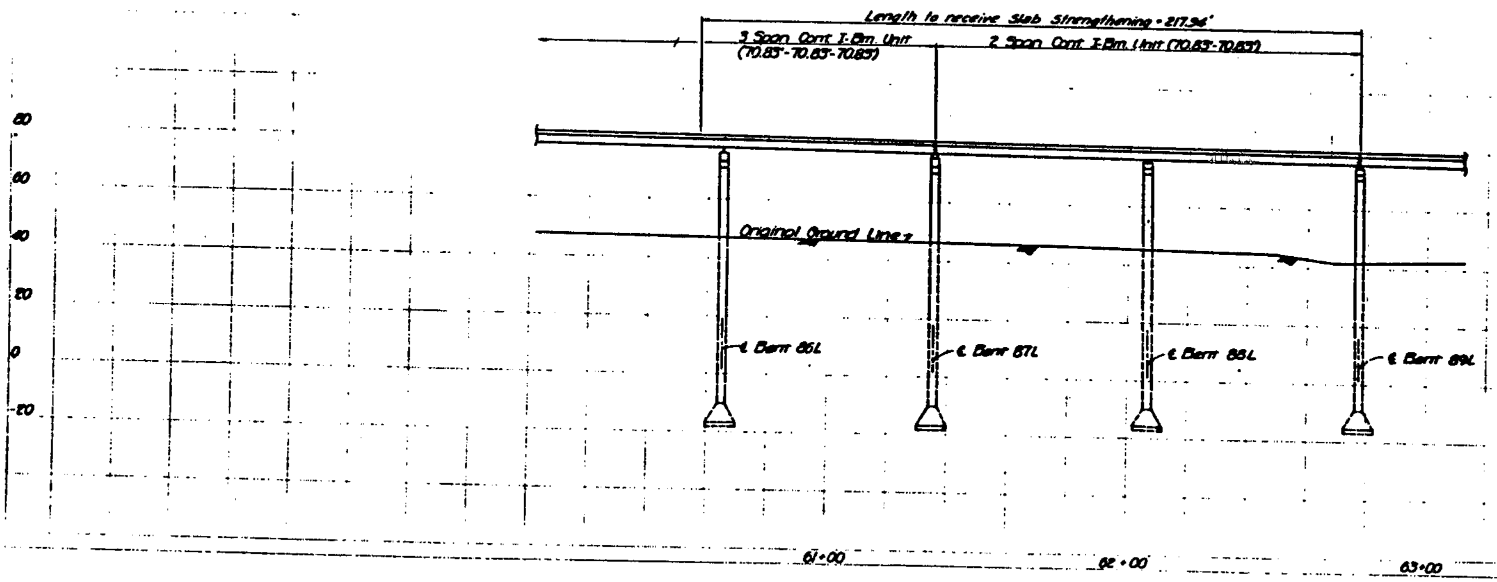


TYPICAL CROSS SECTION

Note:
Existing structure constructed in 1981 under Project I-45-1(25)49 Control 500-3-78

ESTIMATED QUANTITIES *		
Item	Unit	Quantity
Structural Steel (HYC)	Lb	73,600
Epoxy Grout	CY	5.7

* The estimated quantities shown above for information only. All work performed and materials furnished under this project will be paid for at the lump sum bid for "Slab Strengthening System - Structure No. 97."



TEXAS HIGHWAY DEPARTMENT
HOUSTON URBAN PROJECT
I-45 NORTH FREEWAY

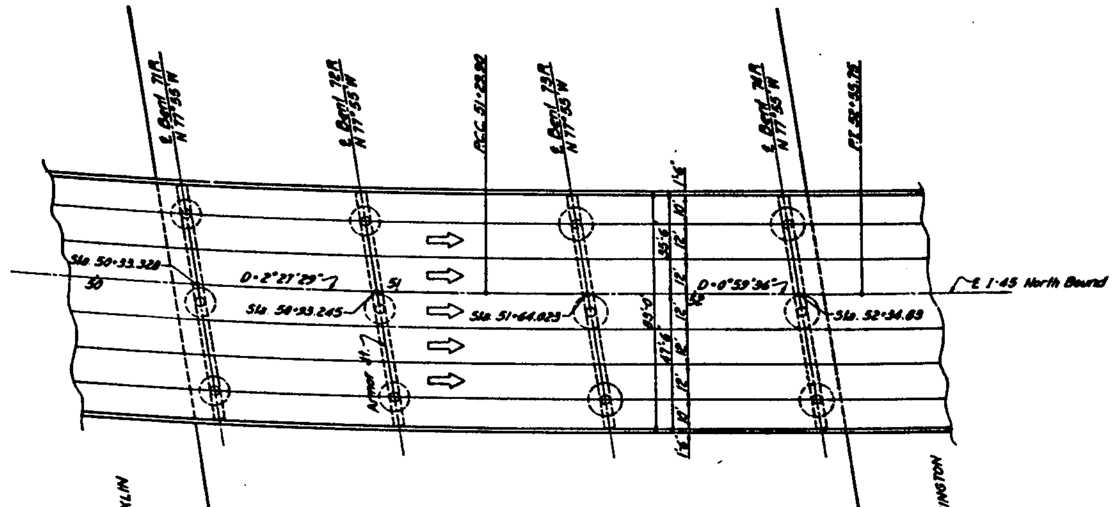
LAYOUT 4

SLAB STRENGTHENING SYSTEM

STRUCTURE NO. 97

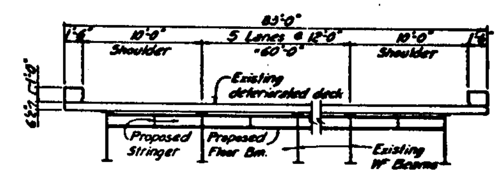
I45-1(144)048

12 HARRIS 500 3 4



PARTIAL PLAN - EXISTING STRUCTURE NO. 98

I-45 NORTHBOUND

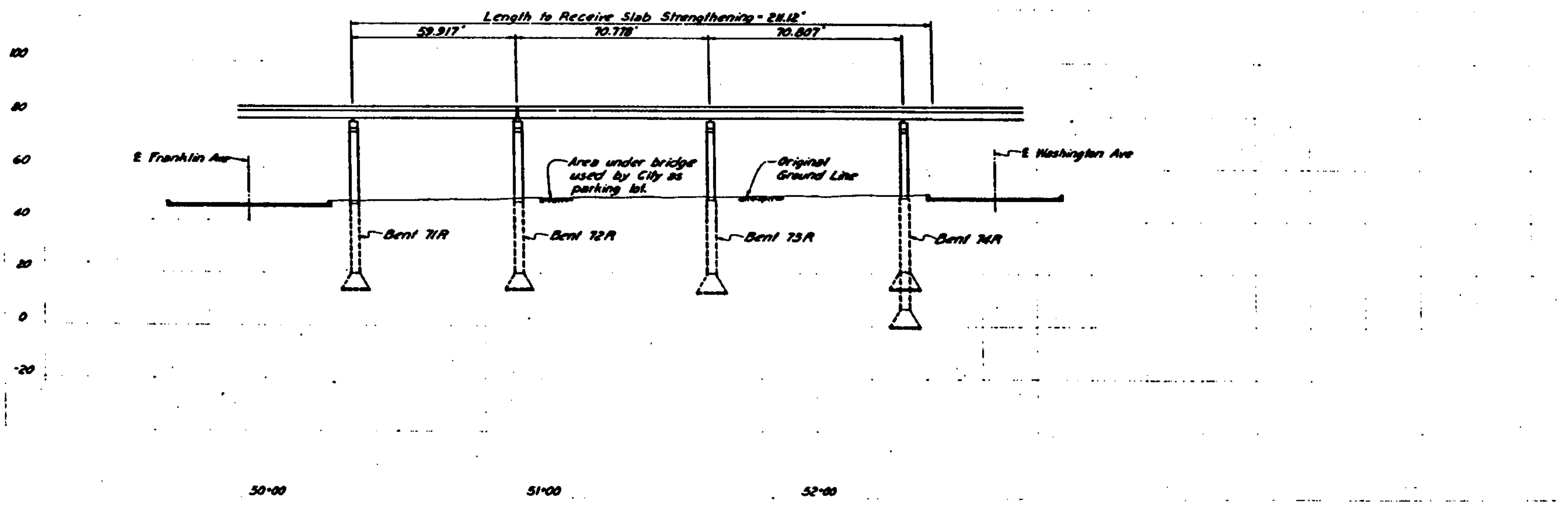


TYPICAL CROSS SECTION

Note:
Existing structure constructed in 1961 under Project I-45-1(25)49 Control 500-3-74.

ESTIMATED QUANTITIES		
Item	*Unit*	*Quantity*
Structural Steel (HYC)	*Lb*	*85,900*
Epoxy Grout	*CY*	*4.3*

• *The estimated quantities shown above for information only. All work performed and materials furnished under this project will be paid for at the lump sum bid for "Slab Strengthening System - Structure No. 98."*



TEXAS HIGHWAY DEPARTMENT
HOUSTON URBAN PROJECT
I-45 NORTH FREEWAY

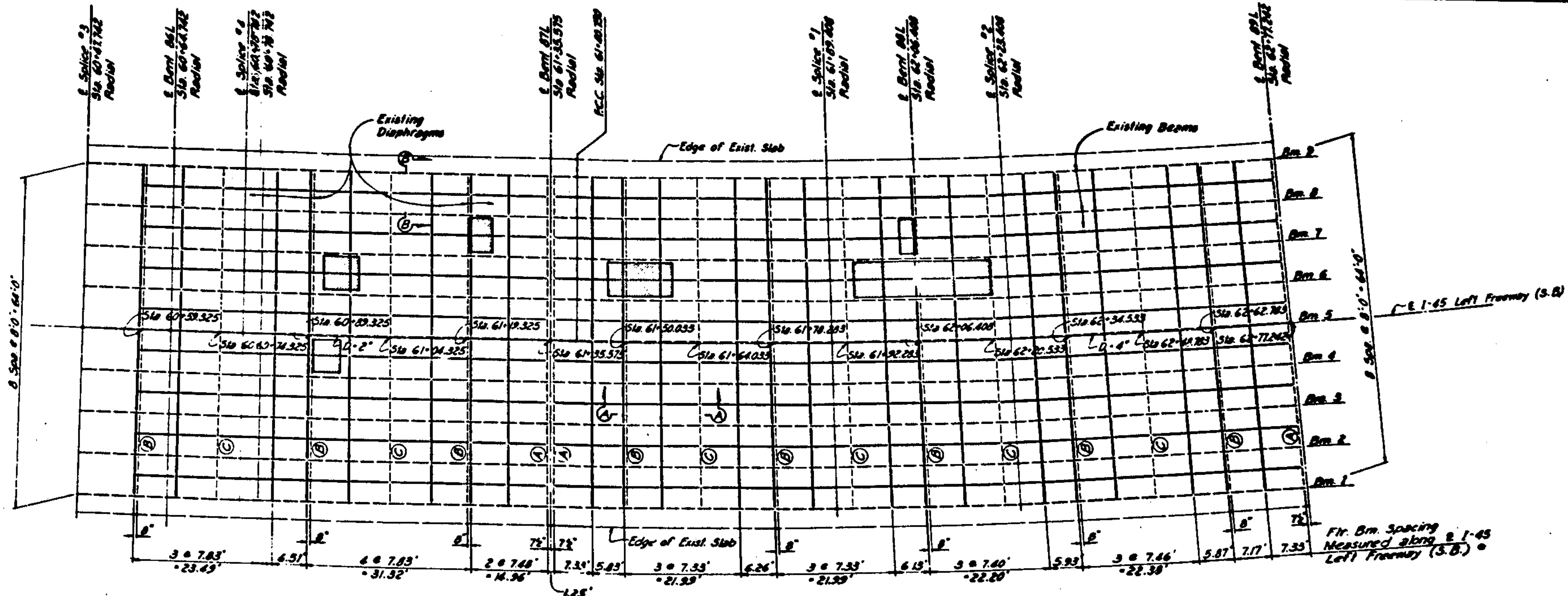
LAYOUT 5

SLAB STRENGTHENING SYSTEM

STRUCTURE NO. 98

C.J.K.
S.L.I. I45-1(144)048 I-45

12 HARRIS 500 3 224 5



* Verify lengths in the field.
Existing Type Ⓒ diaphragms used in place of floor beam.

FRAMING PLAN - STRUCT. NO. 97
I-45 SOUTHBOUND

Note:
☐ Shaded areas indicate locations where deck repairs have been made using steel plates at under side of slab.

Note:
Circled letters Ⓐ Ⓑ & Ⓒ indicate types of existing diaphragms.

TEXAS HIGHWAY DEPARTMENT
HOUSTON URBAN PROJECT
I-45 NORTH FREEWAY

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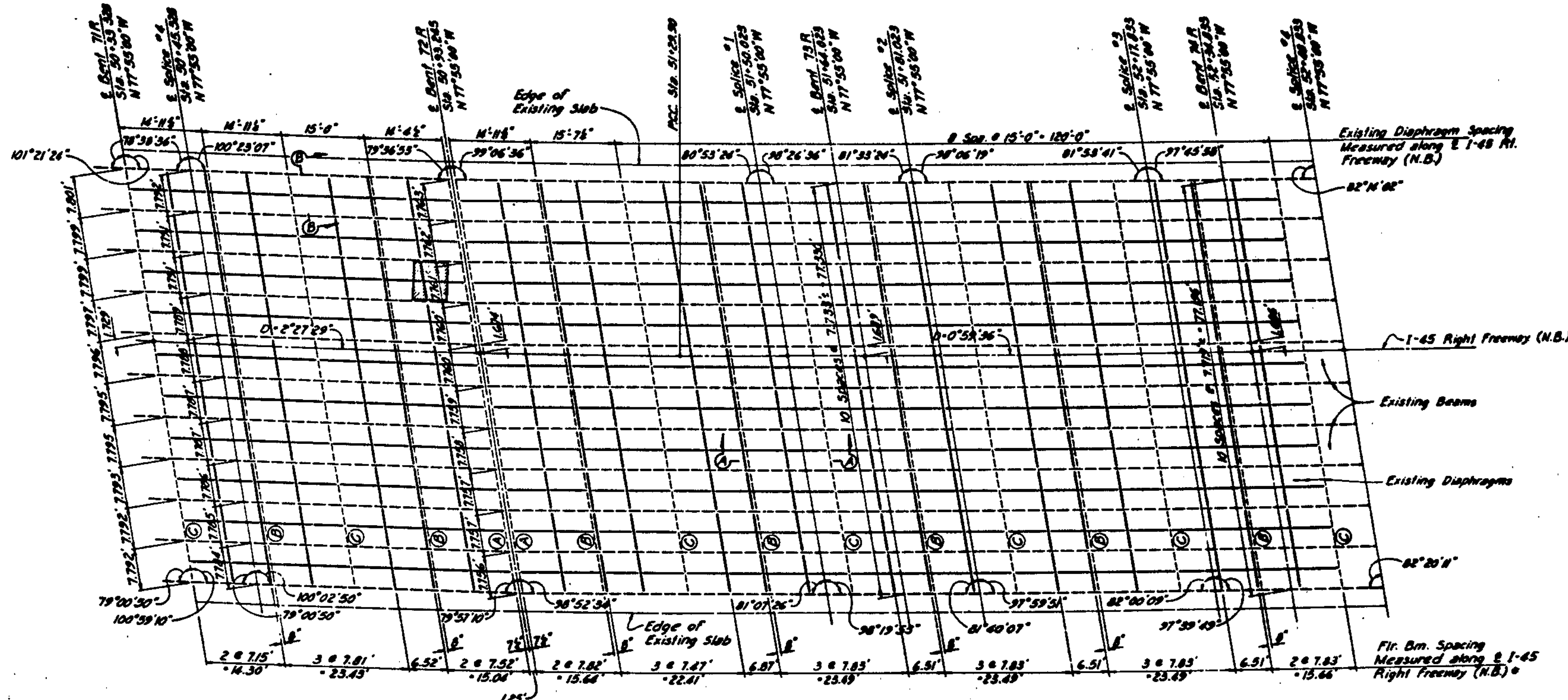
FRAMING PLAN
SLAB STRENGTHENING SYSTEM

STRUCTURE NO. 97

DRAWING ORIGINAL	DATE AUG. 1973	FEDERAL PROJECT NO. I45-1(144)048	I-45
COUNTY HARRIS	CONT. 500	SECT. 3	JOB 224 SHEET 6



FRAMING PLAN-STRUCT. NO. 98

I-45 NORTHBOUND

* Verify lengths in the field.
Existing type Ⓒ diaphragms used in place of floor beam.

Note:
☐ Shaded area indicates existing deck repair with steel plate at under side of slab.

Note:
Circled letters Ⓐ, Ⓑ & Ⓒ indicate types of existing diaphragm.

TEXAS HIGHWAY DEPARTMENT
HOUSTON URBAN PROJECT
I-45 NORTH FREEWAY

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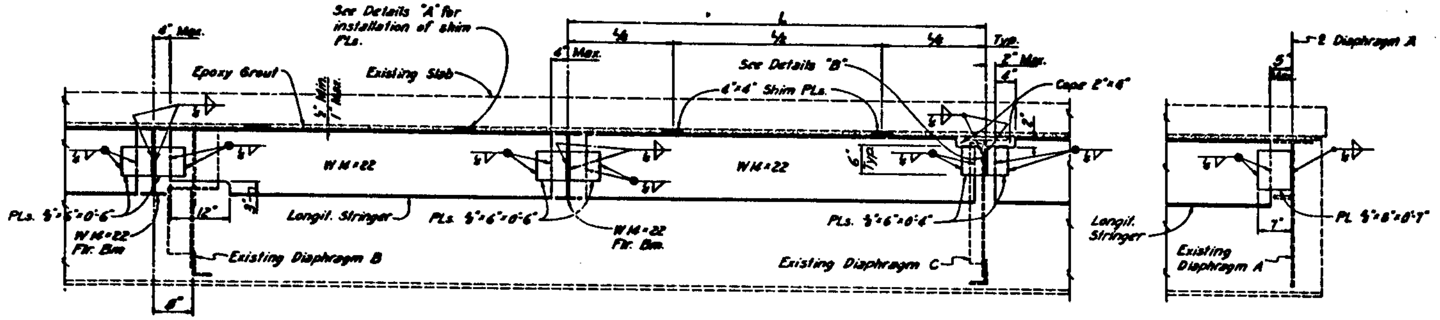
FRAMING PLAN

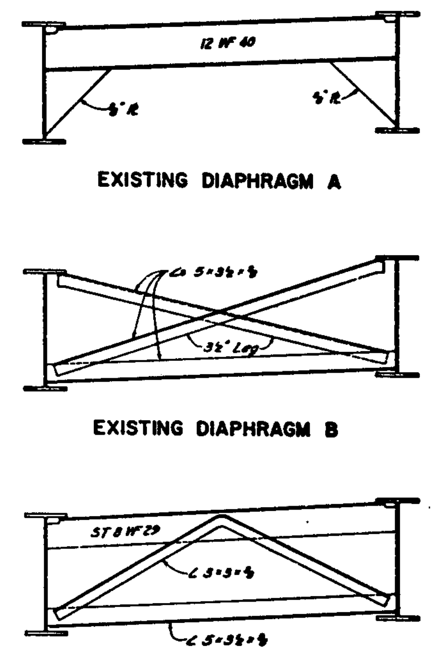
SLAB STRENGTHENING SYSTEM

STRUCTURE NO. 98

DN. B.L.T.	DRAWING ORIGINAL	DATE OCT. 1978	FED. RD. DIV. NO. 6	STATE TEXAS	FEDERAL PROJECT NO. I-45-1(164)048	HIGHWAY NO. I-45
CK. DN. E.J.S.			STATE DIST. NO. 12	COUNTY HARRIS	CONT. SECT. JOB 500 3 224	SHEET NO. 7
DS. C.J.M.						
CK. DS. E.J.S.						



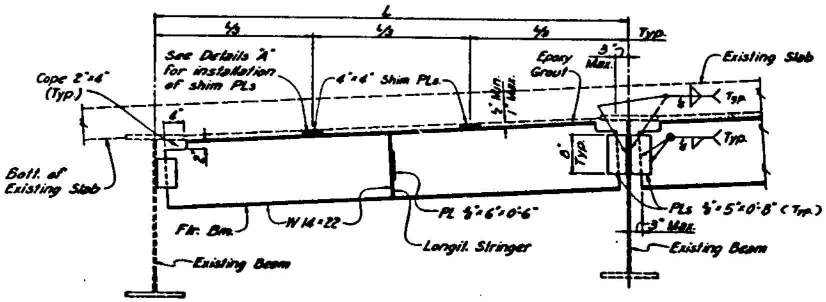
SECTION A-A
(TYPICAL LONGITUDINAL SECTION)



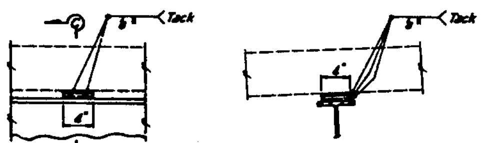
EXISTING DIAPHRAGM A

EXISTING DIAPHRAGM B

EXISTING DIAPHRAGM C

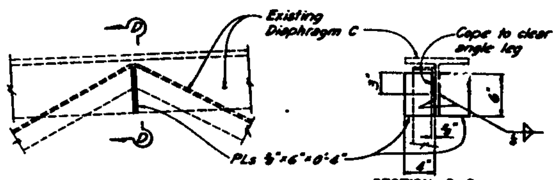


SECTION B-B



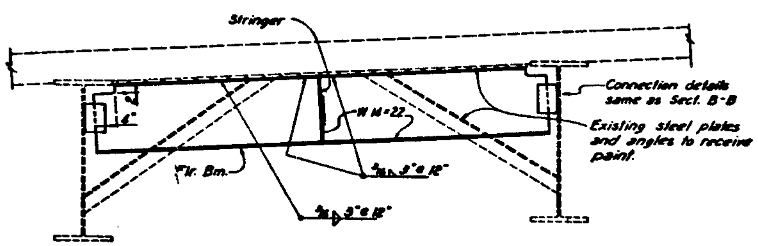
SECTION C-C

DETAILS "A"



SECTION D-D

DETAILS "B"



TYP. SECT. AT EXISTING STEEL PLATE

Notes:

Flat shim plates shall be furnished and placed in increments to provide a tight fit between the bottom of slab and top of supporting beams. The amount of tightness shall be that obtained by driving shim plates with a hand held hammer. In order not to overstress the existing slab, the gap between the slab and beam flange shall not increase more than 1/16" due to driving shims. After shims are placed in several bays, a check shall be made for tightness of shims initially placed, then tack weld to each other and the beam flange.

TEXAS HIGHWAY DEPARTMENT
HOUSTON URBAN PROJECT
I-45 NORTH FREEWAY

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DETAILS

SLAB STRENGTHENING SYSTEM

STRUCTURE NO. 97 & 98

DN. S.L.T.	DRAWING	DATE	
CK. DN. E.J.S.	ORIGINAL	SEPT. 1974	
DW. C.J.M.			
CK. DW. E.J.S.			

FED. RD. DIV. NO.	STATE	FEDERAL PROJECT NO.	HIGHWAY NO.
6	TEXAS	I 45-1(144)048	I-45

STATE DIST. NO.	COUNTY	CONT.	SECT.	JOB	SHEET NO.
12	HARRIS	500	5	224	8

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APPENDIX B

SPECIAL SPECIFICATION FOR EPOXY GROUT

TEXAS HIGHWAY DEPARTMENT

SPECIAL SPECIFICATION

ITEM 4089

EPOXY GROUT

4089.1. Description. This item shall consist of a two-component, 100%-solids, epoxy-resin system mixed with a round grain-silica sand to form a flow resistant grout for filling a one-half inch to one inch space between the underside of a concrete bridge deck and the top flange of steel I-beams to obtain uniform load transfer.

4089.2. Materials. Unless otherwise indicated, tests shall be performed in accordance with AASHTO T 237-731, "Method of Test for Epoxy Resin Adhesive".

(1) Epoxy Binder Properties.

(a) The ratio of resin and hardener components to be mixed together to form the finished binder shall be either 1 to 1 or 2 to 1 by volume.

(b) All fillers, pigments and/or thixotropic agents in either the epoxy resin or hardener component must be of sufficiently fine particle size and dispersed so that no appreciable separation or settling will occur during storage. The components must be free of lumps, skinning and/or foreign material.

(c) The binder shall not contain volatile solvents.

(d) Binder Properties When Mixed.

Pot Life at 77° F., minimum - 26 minutes

Set Time:

At 77° F., maximum - 5 hours
At 60° F., maximum - 8 hours

Thixotropy:

The mixed binder shall not evidence any flow at either 77° F. or 120° F.

Consistency:

The mixed binder shall have a gel-like consistency but must be sufficiently fluid that when mixed with equal parts by volume of round grained silica sand, a workable mix will be obtained.

(e) Binder Properties When Cured.

Adhesive Shear Strength, minimum - 2800 p.s.i.

Water Gain, percent by wt., maximum - 0.20%

Shore Durometer Hardness:

At 77° F., maximum - 90
At 120° F., minimum - 65

(Determined by ASTM D 2240 using a 10 second time interval)

(2) Sand: The sand used shall be a round grain 30 mesh silica sand (at least 98% passing a No. 20 U.S. Standard Screen and retained on a No. 30 U.S. Standard Screen). The sand shall be clean and absolutely dry when mixed with binder.

(3) Epoxy Binder - Sand Mixture (Epoxy Grout).

(a) The grout formed by mixing equal parts by volume of epoxy binder and sand shall have a good troweling consistency.

(b) Test for Flow or Sag.

The epoxy grout shall satisfy the following laboratory test for flow or sag:

Immediately after mixing, a three inch by six inch by one-half inch thick volume of grout shall be applied on a smooth, clean steel panel and the panel and grout shall be placed in a vertical position with the six inch dimension vertical. The grout must not evidence any flow or sag. The ambient temperature and initial temperature of the materials shall be 77° ± 2° F. for this determination.

4089.3. Application of Epoxy Grout.

(1) Surface Preparation. Remove all dust, laitance, grease, curing compounds, impregnations, waxes, and other foreign particles and disintegrated material. Surface must be dry and sound.

(2) Installation. The epoxy grout shall be placed in such a manner as to completely fill the gap between the bottom of slab and top of steel beams.

Placement of epoxy grout shall be completed within the pot life of the material.

4089.4. Measurement and Payment. No measurement for payment will be made under this item. All materials, labor, equipment, methods and incidentals required by this item shall be considered subsidiary to the various bid items in the contract.